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REMEDIATION OF DEEP UNCONTROLLED FILL USING DYNAMIC COMPACTION

By

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A thesis submitted in fulfilment of the requirements for the degree of Doctor of Philosophy School of Civil Engineering University of Sydney

2013

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ABSTRACT

This thesis is based on an industry project examining the geotechnical remediation of a backfilled quarry for residential and commercial development. The study involved two phases, namely: the characterisation and settlement characteristics of the fill; and the implementation of dynamic compaction to render the site suitable for development.

The first part of the thesis is focused on the mechanism and magnitude of settlement of the fill, in particular, collapse settlement. Collapse settlement is a phenomenon that has been documented around the world. Houston *et al* (2003) noted that collapse settlement occurs in both natural deposits and man-made fills that have been placed in a loose condition.

The relationships and case histories presented in the literature do not extend to presenting quantified relationships between engineering properties (such as density or stiffness) and the magnitude of collapse settlements (or strains). Authors such as Lawton *et al* (1989), Charles and Watts (2001) and Charles (2003) have noted relationships between collapse potential and density, but did not specifically quantify the relationship.

Therefore the objective of the first part of this thesis has been to identify the mechanisms involved in the collapse of soil, with the focus then being on quantifying the magnitude of the collapse with respect to the density ratio of the soil. The results of the study enable the development of a detailed understanding of the mechanism involved in the collapse of the fill material and also relationships between in situ wet density and collapse strain. These relationships in turn allow quantification of the amount of collapse strain to be calculated, from which the resultant landform settlement profile can be estimated.

A process for compacting the fill (Dynamic Compaction) was adopted to mitigate the potential effects of collapse settlement, and becomes the focus of the second part of the thesis.

The available literature on dynamic compaction is primarily focused on the depth of improvement generated from a single drop point. However, in the field dynamic compaction is undertaken in an interlocking grid pattern. The relationship between the grid spacing, number of blows and depth of improvement has not been previously explored and is a shortcoming of the current state of knowledge. This thesis presents the results of full scale trials that have been undertaken to quantify the effects of changing the grid spacing and number of blows forming part of the dynamic compaction process. As a result

of the trials, new relationships are presented for assessing the effect of dynamic compaction and designing the number of blows and grid spacing of dynamic compaction.

Furthermore, the literature on dynamic compaction has solely been focused on the compaction or settlement resulting from dynamic compaction; however none of the literature presents information about the near-surface disturbance, or heave, that also occurs. The results of full scale trials have been used to assess the effect that dynamic compaction has on the near-surface materials, or "Heave Zone". The depth and extent of the Heave Zone have been quantified, and modified bearing capacity theory has been used to estimate the heave resulting from dynamic compaction.

Historical relationships have been presented for the depth and magnitude of settlement resulting from dynamic compaction. Based on the results of full scale trials, the previous empirical relationships are refined with amendments to established relationships provided to increase the knowledge in the field of dynamic compaction.

ACKNOWLEDGEMENTS

This research forms part of an industry project examining the remediation of a backfilled quarry in Western Sydney. I am grateful to the Penrith Lakes Development Corporation for giving me permission to use the data collected on site as the basis for this research.

I am also very grateful to my supervisors at the University of Sydney for their support during the tenure of this thesis. Professor John Small was initially overseeing the thesis; and Professor David Airey took over from Professor Small after his retirement. I am particularly grateful to Professor Airey for taking the time to direct my curiosity and see how much more we could get out of the data. I would also like to thank Ross Barker for his support in the laboratory, good humour and dedication to the students at the University of Sydney.

Undertaking the research on a part time basis would not have been possible without the support of Coffey Geotechnics Pty Ltd (my employer). In particular, Sukumar Pathmanandavel and Emeritus Professor Harry Poulos provided the initial encouragement for me to undertake the thesis. They also provided me with words of encouragement and a quiet chat to keep me going, without which this thesis would not have been completed. Coffey has also established the Specialist Technical Apprenticeship Scheme (STAS) through which I have had ongoing assistance and inspiration to complete this thesis. There are many, many colleagues at Coffey who have helped through different ideas, discussions and encouragement that have help along the way and for that I am thankful.

I would also like to thank Dr Peter Redman and Mr Greg Hackney. Dr Redman and Mr Hackney were integral in the initial stages of the research and challenged me to follow my curiosity for both the collapse potential and implementation of dynamic compaction.

Thanks also to my friends and family who provided support and encouragement during the research. The encouragement over the years has helped immensely, particularly during periods where I thought of ceasing the research.

Finally, I would like to thank my girlfriend, Simone. Without her support and inspiration this simply would not have been completed.

PUBLICATIONS ARISING FROM THE RESEARCH

The following publications have been submitted as part of this research:

- Moyle R. A, and Burgess P. B (2006), "Identification and Treatment of Areas Susceptible to Tunnel Erosion", Australian Geomechanics Journal, Volume 41 No 1 March 2006 pages 89 – 98.
- Moyle R. A., Hackney G. A. and Redman P. G. (2009), "Mechanism and Magnitude of Collapse Settlement in Sands and Clays", 4th Asia-Pacific Conference on Unsaturated Soils, November 2009, Newcastle Australia, pages 359 – 364.
- Moyle R. and Redman P. G. (2009), "Optimization of Dynamic Compaction Processes – A Case Study", Ground Improvement Technologies and Case Histories, December 2009, Singapore, pages 739 – 746.
- Moyle R. A. and Redman P. G. (2010), "Assessment of Ground Surface Profile Resulting from Collapsing Soils"; 3rd International Conference on Problematic Soils, April 2010, Adelaide.
- Moyle R. A. and Turner R. M. (2012), "Assessment of Grid Spacing for Dynamic Compaction", ISSMGE International Symposium on Ground Improvement, June 2012, Brussels
- Airey, DW, Nazhat, Y, Moyle R and Avalle, D. (2012) Dynamic compaction Insights from laboratory testing, International Conference on Ground Improvement and Ground Control (ICGI 2012), Wollongong, November 2012
- Moyle R. and Airey D. (2013), "Assessment of Ground Vibration from Ground Treatment Techniques", submitted to Australian Geomechanics Society Journal.
- Moyle R., Airey D., Peiris A. and Redman P. (2013), Use of Topographic Survey in Dynamic Compaction, submitted to Ground Improvement Journal.

CHAPTER ONE

INTRODUCTION

1 INTRODUCTION

1.1 Background

This thesis is based on an industry project examining the geotechnical remediation of a backfilled quarry site in Western Sydney. The quarry site consists of approximately 1,935 hectares that has been in operation since the 1960s; and is the largest quarry in Australia. An aerial photograph of the scheme is presented in Figure 1-1.



Figure 1-1. Aerial Photograph, 2009 (Penrith Lakes Development Corporation)

At the time the works were undertaken, the quarry site was active with the resource consisting of a gravel and sand deposit overlain by Alluvial silty sand and silty clays.

The natural, "pre-quarry", subsurface profile is presented on Figure 1-2.



Figure 1-2. Natural Subsurface Profile

In order to access the gravel and sand for quarrying, the Alluvial deposits are excavated and trucked to previously quarried parts of the site. By reference to Figure 1-1, the quarrying activities are being undertaken in the northern part of the site with fill being placed along the eastern edges of the central lake.

With the quarrying near completion, the area is being reconstructed to include a series of lakes and parks with some residential development.

Over the years of operation, the placement of fill to reconstruct the landform has been undertaken by a variety of methods, typically denoted by the equipment used at the time the landform was constructed.

Where a landform was constructed using truck dumping and spreading of the fill, the landform typically consists of thick layers, which are up to 1m thick. The upper, approximately, 300mm of each layer was generally of higher density and stiffness due to vehicular trafficking during placement, and the remainder of the layer received little or no compaction hence is in a reasonably loose state (and of low stiffness).

Where the landform was constructed using scrapers, it generally comprises thin layers of fill. This is a result of a physical limitation of the scrapers in placing fill, in that scrapers are not designed to allow fill to be placed in layers thicker than about 300mm. This results in a more consistent thickness of each layer of fill placed and also the density and stiffness of each layer is more consistent.

The variability of the fill placement methods has a significant impact on the performance of the landform, in particular for residential development.

This thesis was based on the issues involved in remediating areas of landfill for residential and commercial development. In order to remediate the landform, the study involved two phases:

- 1) Characterisation of the fill and the evaluation of its long-term performance; and,
- 2) Implementation of a remediation strategy to render the site suitable for development.

1.2 **Objectives**

The first part of the thesis is concerned with the mechanism of settlement of the fill, in particular, collapse settlement. Collapse settlement is a phenomenon that has been documented around the world. Houston *et al.* (2003) noted that collapse settlement may occur in recently deposited soils, such as alluvial or colluvial deposits in arid to semi-arid environments. The phenomenon has also been described in man-made fills that have been placed in a loose condition (Houston *et al.* 2003). The focus of this study is on assessing the mechanisms involved in the collapse of soil, and quantifying the magnitude of the collapse strain. Upon quantifying the collapse strain, further work will be described to establish a relationship between the density ratio of the compacted soil, the moisture content and the collapse strain.

A process for compacting the fill (dynamic compaction) was adopted to mitigate the potential effects of collapse settlement. The reasons for the selection of dynamic compaction and the trial of other forms of ground improvement were investigated in more detail in the industry project; but only dynamic compaction is considered herein.

Hence, the second part of this dissertation focuses on the use of dynamic compaction to remediate the soil and reduce the potential for collapse settlement to occur.

Generally, dynamic compaction involves dropping a weight from a predetermined height in a coordinated grid pattern over the treatment area. The improvement in soil behaviour due to

dynamic compaction depends upon factors such as the grid pattern and spacing, the tamper shape and weight, the number of passes and the number of drops used at each grid point.

The issues related to using dynamic compaction to remediate an uncontrolled fill site include the following:

- Numerous authors (Chow *et al.*, 1992 a and b, for example) discuss the "densification" produced by dynamic compaction; however they do not measure the density of the subsurface profile; rather they choose to infer the density from either CPT or SPT results; or quantify densification through a parameter such as friction angle. By doing this, assumptions are introduced that have the potential to lead to erroneous conclusions or limit the interpretation to a particular soil type.
- A considerable focus of the attention of researchers to date has been on the effect of dynamic compaction of loose sands. This has some practical limitations in that dynamic compaction is not only used on loose sands, but is also being used on uncontrolled fills that contain a variety of materials with a range of *in situ* densities.
- A limitation of the numerical modelling to date has been the modelling is primarily focused on the effect of dynamic compaction at a single drop location, whereas dynamic compaction is typically performed in a series of two to three passes. Hence the spacing between drop locations and the number of drops at each location are significant design considerations.
- Dynamic compaction studies have primarily been focused on the amount of settlement induced; however, the dynamic compaction process can also induce a significant amount of heave. The amount of heave induced by dynamic compaction has considerable impact on urban developments that incorporate shallow footings.

The specific objectives for this thesis are as follows:

- 1. <u>Part 1 Understanding and Treating Fill</u>
 - A. Identify the mechanisms involved in collapse settlements, and quantify the magnitude of the collapse with respect to the density and moisture content of the material.
 - B. Develop a methodology for assessing accurately the *in situ* density of the soil.
 - C. Develop a methodology for assessing the effect on the ground surface profile, and buildings, of a collapsible zone occurring within the fill.

2. Part 2 – Understanding Dynamic Compaction

- A. Develop methodologies for assessing the energy required for dynamic compaction, based on the results of a full-scale trial of dynamic compaction on a subsurface profile comprising uncontrolled fill. The energy requirements would be:
 - The grid spacing; and,
 - The number of blows at each drop location.
- B. Identify theory for estimating the effect of ground heave resulting from dynamic compaction, and calibrate the theory with the results of a full scale dynamic compaction trial.
- C. Review literature and provide further refinement of historic empirical relationships regarding the depth and magnitude of settlement induced by dynamic compaction based on the results of the full scale trial.
- D. Assess whether dynamic compaction has been successful in compacting the ground to reduce the impact of voids in the fill which possess the potential to collapse.

1.3 **Outline**

This dissertation presents a review of the literature in Chapter 2. The literature review initially focuses on collapse strain of soil upon inundation in terms of the mechanism involved in the process of collapse and the magnitude of the settlement recorded. The literature review then considers dynamic compaction, in particular on methods of assessing the effectiveness of dynamic compaction as a remediation method. Various case studies of dynamic compaction are also presented.

Part 1 of the objectives is addressed in Chapters 3 and 4 of the thesis.

In Chapter 3, the results of site investigations undertaken to characterise and classify the fill on site are presented. The main emphasis in this chapter is on understanding the mechanism and magnitude of collapse settlement within the fill and calibration of field testing in order to assess the potential for collapse settlement. The methodology for assessing the effect on a landform subject to collapse settlement is presented in Chapter 4. The methodology is based on empirical results, with design plots developed for the expected performance of the landform.

Part 2 of the objectives is then presented in Chapters 5 and 6.

The results of a full-scale dynamic compaction trial are presented in Chapter 5. The focus of Chapter 5 is on the results of the trial as well as studies undertaken to quantify the effect of varying the energy delivered during the dynamic compaction process.

The effects of dynamic compaction are critically reviewed in Chapter 6. The effect of dynamic compaction is divided into three areas: namely the Heave Zone, the Settlement Zone and the Undisturbed Zone. Chapter 6 provides a review of the depth and effect of dynamic compaction within each of these Zones.

The final chapter (Chapter 7) presents a summary of the outcomes from this work and identifies areas for further research.

CHAPTER TWO

LITERATURE REVIEW

2 LITERATURE REVIEW

2.1 Introduction

The method of remediation of a landform is dependent upon the end land use requirements and the existing conditions of the subsurface materials. For an urban development, the settlement performance of the subsurface profile is generally the governing factor. Therefore, an understanding of the settlement characteristics of the fill material is required in order to develop an applicable remediation strategy.

As noted in Chapter 1, the remediation strategy adopted for this site was dynamic compaction; which is a widely used ground improvement technique originally developed in France circa 1977.

The Literature Review has been undertaken in two parts, in line with the objectives detailed in Chapter 1, which include:

- 1. Part 1: Understanding the Fill:
 - A. To identify and examine the settlement characteristics of fill materials.
 Particular focus is on previous works carried out in relation to collapse settlement, or the realignment of the soil structure, upon inundation;
 - B. To review published data and relationships between collapse settlement and field testing; and,
 - C. To examine innovative techniques of examining the soil microstructure, namely: the Scanning Electron Microscope.
- 2. Part 2: Understanding Dynamic Compaction
 - A. To identify state of the art techniques in assessing the effect of dynamic compaction;
 - B. To review the modelling of dynamic compaction and quantification of the ground response to dynamic compaction; and,
 - C. To review case histories where dynamic compaction has been undertaken in collapsible soils in areas for residential development.

Finally, a summary is presented of the works carried out previously and where there is potential to extend the current knowledge base in the assessment of collapse settlement and dynamic compaction.

2.2 Settlement Characteristics of Fill Material

2.2.1 General

The settlement characteristics of fill material have been investigated predominantly from the perspective of the United Kingdom and the United States of America, where monitoring has been carried out in backfilled mines and quarries over an extended period of time. In Australia, the performance of deep-filled sites is becoming an increasingly important issue as more residential and commercial developments are being planned and/or built on sites with a significant thickness of fill.

Leigh and Rainbow (1979) suggested the complexity of the behaviour of fill materials in their work presenting the results of monitoring backfilled opencast mine sites within the United Kingdom. They postulated that the settlement of fill material is governed, in part, by the crushing or destroying of the contact points between soil particles. The breakdown of the contact between soil particles is highly dependent on the material used in the backfilling process, and the thickness of the fill (or overburden pressure). In addition, Leigh and Rainbow (1979) noted that the inclusion of water would be the most important extraneous factor that affects the settlement performance of the fill material.

An example of the monitoring of deep fill performance, is the work carried out by Charles and Burford (1993) who presented the settlement performance of a backfilled coal mine at Horsley (in the United Kingdom) that was monitored from 1973 until 1992 with a series of settlement plates and extensometers.

The work by Charles *et al.* (1993) showed that an increase in the groundwater level caused significant settlement (as much as 2% strains) of the loosely backfilled mudstone and sandstone fragments. In addition, Charles *et al.* (1993) reported that a significant amount of settlement occurred in the upper 10m of the profile which was not affected by the change in groundwater levels, thus leading Charles *et al.* (1993) to note the difficulties in predicting the magnitude and rate of settlement of backfilled sites, predominantly governed by collapse settlements and settlement under self-weight.

2.2.2 Collapse Settlement

The term "collapse settlement" or "hydroconsolidation" is used to describe the process whereby the soil structure is realigned during inundation. The realignment of the soil structure is a rapid process that may result in significant and rapid settlement of the soil.

The following description of collapse upon inundation was provided by Charles (2003): "Partially saturated fills may undergo a reduction in volume known as collapse compression when inundated or submerged for the first time. This compression, which is caused by an increase in water content, can occur without any increase in loading. Most types of partially saturated fill are susceptible to collapse compression under a wide range of applied stress if they have been placed in a sufficiently loose and/or dry condition. When this phenomenon occurs subsequent to construction, buildings can be seriously damaged by the settlement of the fill."

Collapse settlement is a phenomenon that has been documented around the world. Houston *et al* (2003) noted that collapse settlement occurs in recently deposited soils, such as alluvial or colluvial deposits in arid to semi-arid environments. The phenomenon has also been described in man-made fills where they have been placed in a loose condition.

The magnitude and rate of collapse settlement presents a significant issue when a development is proposed within these geological environments. The issue has the potential to be exacerbated in the case of residential and commercial developments, where there is an increased likelihood of collapse due to a higher potential for the addition of water into the soil structure. These sources of water include infiltration from rainfall; the rise of the groundwater level; or a leak in a water bearing service or a pool.

The effects of moisture content, density and overburden on collapse settlement were investigated by Lawton *et al.* (1989) via a series of one-dimensional compression tests. They concluded that the compaction variables are complex and not completely understood. Based on the results of their experiments, Lawton *et al.* (1989) concluded that:

- Moderately plastic soils exhibited both collapse and swelling at lower overburden stresses;
- The wetting induced volume change varies inversely with the moisture content prior to inundation;

- Collapse, and swelling of the soil, could be eliminated by compacting the soil at a moisture content at or greater than what Lawton *et al.* (1989) refer to as the Line of Optimums (a saturation of 80%, which is material specific);
- The collapse and swelling potential could also be reduced by compacting the soil to a critical value of density that is proportional to the overburden stress, for a given soil; and,
- The maximum amount of collapse settlement was observed to occur when the soil was inundated at an overburden pressure equal to the compactive prestress.

Lawton *et al.* (1989) noted that there is limited applicability of these results to all soils, however the results are applicable to some degree to all soils.

Authors such as Tadepalli and Fredlund (1991) or Goodwin and Holden (1993) and Hills and Denby (1995) investigated collapse settlements within a theoretical context, and compared these results with experimental observations such as those presented by Lawton *et al.* (1989).

Tadepalli and Fredlund (1991) used the concepts of unsaturated soil mechanics to explain the collapse behaviour, whereby two independent stress-state variables are used to predict the mechanical behaviour of the collapse of unsaturated soils. The two stress-state variables are the net normal stress (σ -u_a) and the matric suction (u_a-u_w); where σ is the total normal stress, u_a is the pore air pressure, and u_w is the pore water pressure. The matric suction changes during saturation, and the total stress does not change during inundation (Tadepalli and Fredlund, 1991).

Tadepalli and Fredlund (1991) presented the following constitutive equations applying to inundation, by assuming the pore-air pressures in the soil will remain unchanged during collapse:

$$\frac{dV}{V_0} = m_2^s d(-u_w)$$
[2-1]

$$\frac{dV_{w}}{V_{0}} = m_{2}^{w}d(-u_{w})$$
[2-2]

where $u_w =$ water pressure

dV = change in total volume;

 $V_0 = initial volume;$

 dV_w = is the change in volume of water;

 m_2^s = coefficient of total volume change with respect to a change in matric suction at a constant normal stress; and,

 m_2^w = coefficient of water volume change with respect to a change in matric suction at a constant net normal stress.

The one-dimensional water flow equation during inundation has been derived by Tadepalli and Fredlund (1991), and is presented in a finite difference form as follows:

$$u_{w}(i, j+1) = u_{w}(i, j) + \frac{\Delta t}{2\Delta y} \begin{bmatrix} \left(c_{v}^{w}(i+1, j)u_{w}(i+1, j)\right) + \left(c_{v}^{w}(i, j)u_{w}(i-1, j)\right) \\ -\left(c_{v}^{w}(i, j)u_{w}(i, j)\right) - \left(c_{v}^{w}(i+1, j)u_{w}(i, j)\right) \end{bmatrix}$$
[2-3]

where $c_v^w = coefficient$ of consolidation

and the variables used in the finite difference method are shown on Figure 2-1.



Figure 2-1. Variables Used in Finite Difference Method (from Tadepalli and Fredlund, 1991)

The results of the prediction of the negative pore-pressure modelled by [2-3], above, and the results of the computed total volume and water volume calculated by [2-1] and [2-2] were compared by Tadepalli and Fredlund (1991) to experimental data. The results of the testing carried out by Tadepalli and Fredlund (1991) indicated that the collapse strain varies inversely, in a linear manner, with moisture content for a particular dry density. A similar such relationship was presented by Tadepalli and Fredlund (1991) for the initial dry density and the collapse strain; that is, for a given moisture content, as the initial dry density increases the collapse strain decreases.

In terms of the matric suction, Tadepalli and Fredlund (1991) postulated that there is a direct relationship between the matric suction and the total volume change for a soil, thus indicating that the coefficient of volume change with respect to matric suction is either constant or decreases with the reduction in matric suction during collapse.

Goodwin and Holden (1993) and Hills and Denby (1995) assessed the collapse potential by focusing on the air voids, rather than the flow of water or matric suction, as presented by Tadepalli and Fredlund (1991). Hills and Denby (1995) proposed a graphical relationship between collapse settlement and air voids content prior to inundation where the collapse settlement increased with increasing air voids content.

It is important to note that Hills and Denby (1995) postulated that the increase in the bulk density of the material is only effective in reducing collapse potential if there is a corresponding decrease in the percentage air voids.

A number of papers have been published considering the issue of collapse settlement, examples of which are Lawton *et al.* (1992), Charles and Watts (1996), Camapum de Carvalho *et al* (1998) and Charles (2003). These studies generally confirm the "global" relationship between inundation, loose fills and collapse strain, that is the lower the density of the material, the higher the collapse strain.

A significant publication regarding the performance of fills was that by Charles and Watts (2003) Building Research Establishment Guide titled *Building on Fill: Geotechnical Aspects*. Charles and Watts (2003) presented a section on the collapse of fills, noting that collapse compression "*often represents the most serious hazard for building on fill*."

Charles and Watts (2003) postulated that collapse settlements can occur through three main processes:

- Weakening of Inter-particle Bonds: this occurs where bonds between soil particles are weakened due to an increase in the moisture content of the soil.
- Weakening of Particles in Coarse Fill: this mainly occurs in rockfills where the material forming the fill loses some strength due to an increase in moisture content.
- Weakening or Softening of Aggregations of Particles in the Fill: this is where the aggregation (or 'clumps') of clayey materials lose strength during an increase in moisture content.

Charles and Watts (2001) presented the results of a series of investigations, whereby collapse compression ranged between 1% and 5% where there was an increase in ground water level; and a collapse strain of between 3% and 7% from downward infiltration of surface water. The magnitude of potential settlement was reiterated by Charles and Watts (2003) whereby they noted that "ground movements due to collapse compressions may be large, and vertical compressions of the order of 3% to 5% are not unusual".

Charles and Watts (2003) also note that "conventional site investigation methods alone will not provide a satisfactory answer." This sentiment is continued whereby Charles (2003) stated that there was limited applicability of field and laboratory tests to identify collapse potential as these tests are usually reported in terms of overall strength and compressibility, not collapse strain in relation to the density of the soil.

Several case studies are provided in Charles (2003) from locations in the United Kingdom, the United States of America and New Zealand. For these case studies, the depth of fill ranged from 6m to 30m and the settlement, which was assessed to be due to collapse upon inundation, was up to 0.45m. Of interest was the fact that the delay from the time between the fill placement and the time of collapse compression for the selected case studies ranged from 5 years to 246 years, indicating that collapse settlement was not time dependent.

Charles (2008) provided case studies regarding the collapse settlement, and of particular note was a backfilled mine in Ilkeston. The backfill process involved the placement of a medium plasticity clay material using scrapers with little or no additional compaction. Subsequent to the construction of the houses over the fill, including the excavation of

drainage trenches, settlement was recorded within some of the houses. Further tests were carried out comprising the excavation of 3m deep trenches which were filled with water. The results of these tests indicated that settlement occurred when water was filled in the trenches, and continued at a significant rate. The rate of settlement also increased after rainfall events.

Charles (2008) concluded that "poorly compacted opencast backfill placed with little or no control is likely to be in a metastable condition and, irrespective of the age of the fill, there is a risk that some small disturbance, such as an increase in water content, will cause a significant reduction in volume."

Camapum de Carvalho *et al.* (1998) have investigated the effect of inundation on the soil structure and the resultant collapse. Their results are of interest in illustrating the role of microstructure, or soil fabric, in collapse potential. This is consistent with the behaviour of fill material postulated by Leigh and Rainbow (1979).

Camapum de Carvalho *et al.* (1998) showed that the microstructure of a porous clay consisted of micro aggregates of particles bonded together by weak "clay bridges", resulting in a metastable and highly porous structure. These aggregations of particles have also been described by Lawton *et al.* (1992).

Both Camapum de Carvalho *et al.* (1998) and Lawton *et al.* (1992) observed that for low clay content soils (<10% clay content), the clay acts as a binder between silt and sand particles to form a metastable structure. A metastable bond can also exist as a result of the capillary suction in a partially saturated condition (Mitchell 1976). The observation of soil structure by Camapum de Carvalho *et al.* (1998) indicated the potential collapse mechanisms in clay as being the softening of the clay binding upon inundation at aggregate contacts and, in sands, the collapse mechanism was the loss of capillary suction.

Thompson (2003) described a case history of inundation collapse settlement that occurred some 30 to 90 years after residential houses were constructed on a backfilled site. The cause of the inundation was found to be a leaking water pipe in an adjacent road.

A further case history was presented by Kropp *et al.* (1994), where a project in San Francisco, California, was constructed over deep compacted fill materials, and damage occurred to condominiums within months of having being constructed. Most of the

damage occurred over large variations in fill depth. Problems included localised water ponding, sink holes, settlement around storm drains, cracks in pavement surfaces and in the ground surface. In several cases, most of the settlement occurred within the first two years. Wetting of the soil and mud jacking were considered feasible to mitigate the damage, however, there were no reports on the remediation. It was concluded that the settlement was primarily caused by wetting induced collapse of the deep soil fill.

Dudley (1970) made the point that collapse settlement is not just a fill problem and that collapse settlement naturally occurs in arid and other environments where the soils are dry, such as in South Africa, Africa, Australia, India and the United States of America. The types of natural soils in which collapse settlement can occur, or has been known to occur, includes aeolian, alluvium, colluvial, loess and residual soils. For one particular case in California, it was reported that "the clay content appears to be critical, maximum subsidence occurs where the clay amounts to about 12% of the solids. Below 5% there is little subsidence and above 30% the clays swell. In between there are many cases where the soil would swell under a small load and collapse under a large load."

The relationships and case histories presented in literature discuss the mechanism of collapse settlements occurring with considering detail about the formation of metastable structures through clay acting as a binder (Camapum de Carvalho *et al.* (1988) for example). However, quantified relationships between engineering properties (such as density or stiffness) and the magnitude of collapse strain have not been presented. Authors such as Lawton *et al.* (1989) have noted relationships between collapse potential and density, but did not quantify the relationship. Charles and Watts (2001) and Charles (2003) also noted there was limited applicability of field and laboratory tests to identify collapse potential as these tests are usually reported in terms of overall strength and compressibility, not collapse strain in relation to the density of the soil.

Further limitations also exist as to quantifying the impact of the collapse potential on the ground surface profile, and the development constructed on soils with collapse potential.

2.3 **Dynamic Compaction**

2.3.1 General

Dynamic compaction is a relatively simple process whereby a crane is used to raise and drop a steel pounder from a predetermined height to impact on the soil causing densification (Lukas, 1992). The mass of the pounders typically ranges up to 40 tonnes, with drop heights as high as 40m.

Figure 2-5 presents a diagrammatic representation of the principles of dynamic compaction:



Figure 2-5. Dynamic Compaction (from Sennet & Nestvold, 1992)

A result of the dynamic compaction process is the formation of near surface craters, typically up to 4m deep depending on the energy and pounder shape used in the dynamic compaction process. An example of the craters resulting from dynamic compaction is shown on the photograph presented as Figure 2-6, below.



Figure 2-6. Photograph of Dynamic Compaction

The depth to which dynamic compaction densifies the soil was originally postulated by Menard and Broise (1975) as follows:

$$D = 0.5 (W \times H)^{\frac{1}{2}}$$
where D = Depth of influence (metres);
W = weight of tamper (tonnes); and,

H = drop height (metres).

Lukas (1992) presented a modification to Equation [2-4] taking into account factors such as cable drag (which reduces the impact energy); the subsurface profile, the contact pressure between the pounder and the ground and the presence of energy absorbing layers. The modified equation from Lukas (1992) is as follows:

$$D = n (W x H)^{\frac{1}{2}}$$
 [2-5]

where n = empirical coefficient.

Lukas (1992) also presented a series of "n values" taking into account the factors mentioned above. The suggested n values are presented in Table 2-2.

Soil Type	Degree of Saturation	Recommended n Value*
Pervious Soil Deposits – Granular	High	0.5
Soils	Low	0.5 to 0.6
Semi-Pervious Soil Deposits.	High	0.35 to 0.4
Index < 8	Low	0.4 to 0.5
Imporvious Doposita Primarily	Uich	Natracommondad
Impervious Deposits Filmarity	nigii	Not recommended
Clayey Soils with Plasticity Index	Low	0.35 to 0.4
> 8		Soils should be at a water
		content less than the plastic
		limit

 Table 2-2.
 Recommended n Value for Different Soil Types (From Lukas, 1992)

* For an applied energy of 34 to 100 ton-ft/ft² (100 to 300 tonne-m/m²) and for a weight dropped using a single line cable with a free spool drum.

Luongo (1992) has identified that the major factors affecting the effective depth of improvement can be categorised into two major groups, namely: *in situ* conditions, such as the subsurface profile and moisture condition etc., and also mechanical conditions, such as equipment limitations and tamper contact pressure.

Lukas (1992) also realised the effect of the subsurface profile on dynamic compaction, noting that only particular deposits are suitable for dynamic compaction, with the most important influence on the effectiveness of dynamic compaction being the ability of the treated soils to dissipate excess pore pressures generated during the compaction process. This is reflected in Table 2-2, in that the depth to which dynamic compaction was compacting the soil is less in semi-pervious and impervious soils compared to pervious soil deposits (represented in Table 2-2 by a lower n value).

In partially saturated clays the impact of tamping causes "shock" pore pressures that are of a relatively high magnitude in a localised area. Due to the large difference in pore pressure between the "shocked" zone and the surrounding area, the pore pressures dissipate quickly, which results in rapid consolidation of the soil. In dry granular soils the impact of the heavy weight can lead to a physical displacement of particles and hence densification of the ground. In saturated sands the pore pressure response can induce liquefaction.

Luongo (1992) presented further guidelines on the depth of influence of dynamic compaction, based on site-specific information, as shown in Table 2-3.

 Table 2-3. Anticipated Depth of Influence for Different Soil Deposits (Luongo, 1992)

Category		Deposit			
		Pervious Semi-Pervious		Partially Saturated	
				Impervious Fill	
	Lower Bound	D = 5.6+0.009WH	D = 1.4+0.016WH	D = 2.0+0.012WH	
General	Average	D = 7.9+0.009WH	D = 3.4 + 0.016 WH	D = 2.6 + 0.012 WH	
	Upper Bound	D = 10.1+0.009WH	D = 5.3 + 0.016 WH	D = 3.3+0.012WH	
Sample Ran	nge	125 <wh<400< td=""><td>150<wh<400< td=""><td>250<wh<400< td=""></wh<400<></td></wh<400<></td></wh<400<>	150 <wh<400< td=""><td>250<wh<400< td=""></wh<400<></td></wh<400<>	250 <wh<400< td=""></wh<400<>	
High	Lower Bound	D = 4.6+0.012WH		Dynamic Compaction	
Water	Average	D = 7.4+0.012WH	Insufficient Data	Not recommended	
Table	Upper Bound	D = 10.1+0.012WH			
Sample Range		125 <wh<400< td=""><td></td><td></td></wh<400<>			
Low	Lower Bound		D = 1.7+0.016WH	Same as for General	
Water	Average	Insufficient Data	D = 3.3+0.016WH	Category	
Table	Upper Bound	_	D = 4.9 + 0.016WH		
Sample Range			200 <wh<400< td=""><td></td></wh<400<>		
Tamper	nper Lower Bound $*D = -0.2 + 0.024 WH$		*D = -0.4 + 0.024 WH	*D = 4.9+0.004WH	
Base Area	Average	*D = 1.5+0.024WH	*D = 1.3+0.024WH	*D = 5.1+0.004WH	
sq.m only	Upper Bound	*D = 3.2+0.024WH	*D = 3.0+0.024WH	*D = 5.4+0.004WH	
Sample Range		200 <wh<325< td=""><td>150<wh<400< td=""><td>250<wh<350< td=""></wh<350<></td></wh<400<></td></wh<325<>	150 <wh<400< td=""><td>250<wh<350< td=""></wh<350<></td></wh<400<>	250 <wh<350< td=""></wh<350<>	

*Because of the small number of samples these equations should be interpreted with caution.

where D = Depth of influence (metres);

W = weight of tamper (tonnes); and,

H = drop height (metres).

It is important to note that the depth of improvement decreases with depth. The depth of improvement is based on the testing undertaken and, at the deepest extent, may only be a small improvement. Furthermore, the near surface layers are often highly disturbed.

Smits and De Quelerij (1989) provided a model for estimating the depth of improvement, assuming the lateral spread of the dynamic load is ignored, and that the near surface layers act as a rigid body. This is illustrated in Figure 2-7.



Figure 2-7. Soil Deformation as a Result of Wave Propagation (Smits and De Quelerij, 1989)

Smits and De Quelerij (1989) derived an equation for the depth of compaction (or the depth of the compaction zone, "Zone B" from Figure 2-7) by considering that the compacted zone (Zone A) acts as a rigid body, and that any settlement during the elastic compression of the soil is ignored (for simplicity). The equation derived by Smits and De Quelerij (1989) is as follows:

$$H = Z_P - Z_S = \frac{m}{\rho_{\text{max}}A} \left[-1 + \sqrt{\frac{\sigma_e + \alpha v_0^2}{\sigma_e}} \right]$$
[2-6]

where H = Depth of compaction (metres);

 Z_p = Depth to the bottom of the compacted zone (as shown on Figure 2-20);

 Z_s = Depth to the top of the compacted zone (as shown on Figure 2-20);

m = Mass of pounder (tonne);

 ρ_{max} = Maximum density of the soil (t/m³);

A = Plan area of pounder (m²);

 σ_e = Vertical stress of the soil at the elastic limit (kN/m²);

 v_0 = Particle velocity (m/s);

$$\alpha = \frac{\rho_{\text{max}}\rho_e}{\rho_{\text{max}} - \rho_e}$$
; and

 ρ_e = Density of soil at elastic limit (t/m³).

Smits and De Quelerij (1989) noted that the density of the soil at the elastic limit is approximately the *in situ* density. Based on a comparison to a case study of a site in the Netherlands, Smits and De Quelerij (1989) concluded that the model represents the effects of dynamic compaction reasonably well.

Comparison between the anticipated depth of improvement postulated by Smits and De Quelerij (1989), Lukas (1992) and Luongo (1992) shows that the depth estimated for the lower bound case (or soils of high saturation) are generally consistent. However, Luongo (1992) predicts depths of improvement at the upper bound range that are typically 2m larger than those predicted by Lukas (1992) for soils of low saturation. This is not unreasonable considering that the dynamic compaction is influenced by a range of factors, and not just the soils' ability to dissipate pore pressures.

A substantial amount of work has been focused on the vertical depth of influence (as presented above), however limited work has been carried out regarding the lateral movement of soils during dynamic compaction. Lukas (1992) presented results of the lateral ground compression as an indicator of the improvement of the ground. The lateral deflection recorded through inclinometers by Lukas (1992) with blow count is presented in Figure 2-8.

Lukas (1992) also confirmed that the maximum degree of improvement occurred at a depth of 1/3 to 1/2 of the total depth of improvement. He also presented an inferred depth of improvement with blow count, based on lateral deflections, which is presented in Figure 2-9. As can be seen from this figure, the majority of the depth of improvement is obtained



within the initial blows of the dynamic compaction.

Figure 2-8. Lateral Deflection vs Blow Count (Lukas, 1992)



Figure 2-9. Depth of Improvement Inferred from Lateral Deflections (Lukas, 1992)

Thilakasiri *et al.* (1996) considered the lateral effects from dynamic compaction by considering the three distinct zones of soil adjacent to the pounder, namely:

- Zone 1 soil under the pounder that undergoes significant vertical deformation and is considered as a moving soil mass;
- Zone 2 soil in the immediate vicinity of Zone 1 undergoing excessive shear deformations that are classified by non-linear models; and,
- Zone 3 soil experiencing limited shear deformations within elastic limits.

These zones are illustrated (in plan) in Figure 2-10.



zone 3

Figure 2-10. Plan View of the Impact Zones (from Thilakasiri et al., 1996)

Thilakasiri *et al.* (1996) presented a numerical formulation of the behaviour of these three zones, considering the compatibility or continuity across each border of the zone. The dynamic model presented by Thilakasiri *et al.* (1996) is shown in Figure 2-11.



Figure 2-11. Dynamic Model (from Thilakasiri et al., 1996)

Generally, Zone 1 has large strains and hence would be modelled by non-linear plastic behaviour. Therefore, Thilakasiri *et al.* (1996) considered the soil elements as a lumped mass with a cross sectional area equivalent to that of the pounder and connected by non-linear springs.

The elements within Zone 2 are represented by two masses of radius r_1 , which are also subject to high strain levels, therefore the behaviour of the elements within Zone 2 are also modelled by non-linear shear stiffness properties. The elements in Zone 2 interact with the elements of Zone 1 through a rigid slider, as indicated in Figure 2-11.

Outside Zone 2, linear elastic relationships are adopted due to the small strain of each element. Therefore to model the shearing resistance of Zone 3, linear spring and dashpot coefficients are adopted. Hence, Zone 3 provides shear resistance to the vertical movement of the elements in Zone 2.

Poran and Rodriguez (1992) also investigated both the vertical and lateral extent of the influence of dynamic compaction. They postulated that the zone of influence may be represented by a semi-spheroid shape with a (horizontal radius) and b (vertical depth), as presented in Figure 2-12 below.



Figure 2-12. Spheroid Approximation of Zone of Influence (Poran and Rodriguez, 1992)

Poran and Rodriguez (1992) presented relationships between the horizontal radius and the vertical depth of the semi-spheroid to the equipment dependent factors. The relationships are as follows:

$$\frac{b}{D} = j + k \log\left(\frac{NWH}{Ab}\right)$$
[2-7]

$$\frac{a}{D} = l + m \log\left(\frac{NWH}{Ab}\right)$$
[2-8]

where j, k, l &m are empirical factors

N = Number of drops;

W = Pounder mass (tonnes);

H = Drop Height (metres);

A = Contact area between the tamper and the soil (metres²)

D = pounder diameter (metres).

The empirical factors j, k, l and m are presented in Table 2-4.

 Table 2-4. Empirical Factors (from Poran and Rodriguez, 1992)

	State of Improvement			
	Unaffected	Slight	Moderate	Strong
Relative Density Dr (%)	25	35	65	85
j	-12.59	-13.22	-15.27	-15.27
k	8.08	7.91	7.79	6.25
1	-2.49	-2.39	-4.25	-4.43
m	1.97	1.90	2.32	1.99

Poran and Rodriguez (1992) proposed a design chart for sandy soils, which is presented in Figure 2-13.


Figure 2-13. Proposed Design Charts for Dynamic Compaction in Sandy Soils (Poran and Rodriguez, 1992)

The above equations, empirical factors and design charts can be used to develop a remediation strategy in the following manner:

- 1) The soil is assessed and the desired depth of improvement, b, is established;
- 2) The mechanical characteristics are determined (W, D and H) and A is calculated;
- The value of NWH/Ab is determined from the design chart presented in Figure 2-13, using the appropriate b/D value. Hence, the number of drops N can be assessed;
- 4) The a/D value is determined from the design chart presented in Figure 2-13; and,
- 5) Hence, an initial grid spacing can be assessed.

Pan and Selby (2002) simulated the effects of dynamic compaction of loose soils numerically, using the computer program ABAQUS©. The numerical model represented an axi-symmetric elasto-plastic finite element model of the soil, with the impact of the drop mass modelled, firstly, by a force-time derived input based on the characteristic deceleration of the mass and, secondly, as a rigid body impacting onto the soil surface.

The modelling behaviour of the soil is based on compression waves (or P-waves) being generated by dynamic compaction which spreads down and laterally from the point of impact. As the wave propagates to greater depths around a larger spherical front, the

energy attenuates. Pan and Selby (2002) postulated that the soil improvement is a function of the compressive waves; the spread of the compression waves defines the zone of improvement.

Pan and Selby (2002) carried out numerical analysis by adopting a uniform soil profile modelled by a finite element mesh. Based on the modelling carried out by Pan and Selby (2002), it appears that the force-time load overestimated the depth of improvement and also the crater depth generated during dynamic compaction, when compared to field trials. The rigid body impact load, however, presented reasonably consistent outcomes with the field trials in terms of both the depth of influence and the crater depth.

Pan and Selby (2002) also investigated the cumulative effect of up to three drops at a single drop location; but they noted that a number of assumptions had to be made which, in this author's opinion, was an over-simplification of the observed conditions on site. This is also confirmed by the results of the model indicating significantly larger depths of influence than otherwise reported in literature.

Chow *et al.* (1992b) investigated the effect of print spacing (distance between drop locations) on ground improvement using dynamic compaction. They presented an approach to estimate the lateral extent of the ground improvement based on assessing the degree of improvement by the increase in friction angle in cohesionless materials. The estimation of the friction angle was derived from empirical correlations with CPT coneresistance.

Chow *et al.* (1992a) presented design curves for sands that relates the change in friction angle at the centre of the grid ($\Delta \phi_c$) and midpoint of the grid ($\Delta \phi_m$), normalised by the change in friction angle underneath the point of impact $\Delta \phi_b$, to the distance from the drop location, S, normalised by the diameter of the pounder, D. The design curves for the centre of the grid pattern and the mid-point of the grid pattern are presented in Figure 2-14.



(b) Middle of One Side

Figure 2-14. Design Curves for Print Spacing (from Chow et al, 1992a)

Gu and Lee (2002) developed a two-dimensional model to represent both the vertical and lateral spread of energy resulting from dynamic compaction, taking into consideration the large deformations induced by dynamic compaction (i.e. the formation of craters).

The finite element modelling presented by Gu and Lee (2002) considers the large strains experienced by the soil by incorporating an updated Lagrangian large-deformation formulation into the analysis code. The impact of the pounder was modelled by considering the pounder as a stiff elastic block with a velocity calculated from a drop height, assuming free fall conditions. An important adaptation of the modelling was that Gu and Lee (2002) configured the density of each element as a variable, thus allowing it to be changed in each time step.

The constitutive model developed by Gu and Lee (2002) is based on the assumption that densification or improvement is caused by transient effective stress increase induced by pounder impulse rather than vibration-induced densification. The behaviour of the repeated drops was considered by Gu and Lee (2002) to be impulsive, rather than cyclic, hence cyclic densification procedures such as the transition from densification to dilatant behaviour was not incorporated into the constitutive model.

Comparison was made by Gu and Lee (2002) between the results of the finite element modelling and centrifuge testing carried out by Oshima & Takada (1998). The results of



the computed and final measured increase in relative density are presented in Figure 2-15 below for three different drop heights and pounder mass combinations.





The results presented above show reasonable correlation between the computed results and the experimental data. Both sets of results indicate that the increase in maximum density occurs at a depth just below the crater and diminishes rapidly with depth. However, Gu and Lee (2002) note that the increase in density is dependent upon the properties of the soil being subject to dynamic compaction.

Gu and Lee (2002) noted that although 40 blows were delivered during the centrifuge modelling and in the finite element modelling, the computed results indicate that the soil underwent no significant further compaction after about 15 blows. This is consistent with the results of Takada & Oshima (1994), Mayne *et al.* (1984) and Poran *et al.* (1992) that indicate that the majority of settlement induced occurs within the first 20 blows.

Gu and Lee (2002) noted that the stress path is near a one-dimensional compression just near the pounder, due to the lateral inertia of the soil. With depth, the lateral inertia reduces and the stress path morphs to that of a triaxial compression. Away from the pounder, the soil is compacted by lateral earth pressure with a stress path comparable to that of triaxial extension. The effect of the pounder base area was investigated during the modelling of Lee and Gu (2004). Gu and Lee (2002) noted that the soil beneath the pounder undergoes a period of one-dimensional compression, thus increasing the pounder size would prolong this period and compact the ground more. Therefore, Gu and Lee (2002) suggested that for a given pounder mass there is an optimum base area.

Lee and Gu (2004) investigated the effect of the pounder base area, by correlating the radius of the pounder with the depth of improvement. Plots of the area-normalised depth of improvement versus radius of pounder for a variety of momentums and energies are presented in Figures 2-16 to 2-18.



Figure 2-16. Area Normalised Depth of Improvement, Energy = 200tm, Momentum = 200tm/s (Lee and Gu, 2004)

The results of Lee and Gu (2004) indicated that there is an optimal radius for a pounder. For example, if the radius is too small the lateral confinement of the soil directly beneath the pounder is only maintained for a very short period of time, thereby limiting the depth over which one-dimensional wave propagation occurs. Conversely, if the radius of the pounder is too large, the impact stress is reduced to a level that limits the depth of improvement.



Figure 2-17. Area Normalised Depth of Improvement, Energy = 200tm, Momentum = 400tm/s (Lee and Gu, 2004)



Figure 2-18. Area Normalised Depth of Improvement, Energy = 400tm, Momentum = 400tm/s (Lee and Gu, 2004)

2.3.2 Assessment of Dynamic Compaction

The Dynamic Compaction process has been extensively evaluated by a variety of authors using a range of techniques. Generally, geotechnical testing such as Standard Penetration Testing (SPT) and Electric Friction Cone Penetration Testing (CPT) are carried out before and after the dynamic compaction process to assess the degree of improvement (for example, Sennet & Nestvold (1992), Luongo (1992)). Further examples of assessing dynamic compaction using typical methodologies are presented in the following subsection ("Case Studies"). This section presents innovative methods for assessing the effectiveness of dynamic compaction.

Poran *et al.* (1992) presented a method for assessing the dynamic stiffness of a soil mass affected by dynamic compaction (based on dry sand). The method integrates acceleration, with respect to time, using measured integration constants.

A plot of the accelerations recorded on the tamper during impact in loose sand is presented in Figure 2-19. As the number of drops increase so does the dynamic stiffness of the sandy material, as reflected by the increase in acceleration and the decrease in the response time.



Figure 2-19. Impact Accelerations Recorded (from Poran et al., 1992)

The recorded accelerations are integrated to estimate the velocity, which in turn is integrated to assess the pounder displacement versus time. The pounder displacement is plotted against the impact stress to assess the effectiveness of the dynamic compaction process.

The impact stress is calculated as follows:

$$p_t = \frac{ma_t}{A}$$
[2-9]

where p_t = the impact stress (kPa);

m = mass of the pounder (tonne);

 a_t = acceleration (metres/second/second); and,

A = contact area.

An example of the impact stress calculated by Poran *et al.* (1992) is presented in Figure 2-20. Also presented on Figure 2-20 is a graphical representation of the calculation of the dynamic settlement modulus (DSM), which is the maximum gradient of the impact stress to the pounder displacement (referred to by Poran *et al.* (1992) as a tamper).



Figure 2-20. Impact Stress vs Pounder (tamper) Settlement (from Poran *et al.*, **1992)** Poran *et al.* (1992) suggest that DSM values were proportional to the rate of densification and the elastic modulus. An expression relating DSM to the elastic modulus was presented, however only limited correlations to the density have been provided.

Mayne and Jones (1983) presented the deceleration response of a pounder during dynamic compaction from a site in Alabama, USA. At the site a pounder weighing 20.9 tonnes (W) and being dropped from 18.3m (H) was adopted. As can be seen in Figure 2-21, the response is essentially a triangular impulse loading (Mayne and Jones, 1983).



Figure 2-21. Deceleration Response during Dynamic Compaction (Mayne and Jones, 1983)

Mayne and Jones (1983) equated the peak dynamic maximum force (F_{MAX}) during dynamic compaction to the pounder mass and drop height as follows:

$$F_{MAX} = \sqrt{\frac{32WHGr_o}{\pi^2(1-\nu)}}$$
[2-10]

where G = Shear Modulus

 $r_o = radius of pounder$

Mayne and Jones (1983) estimated the peak vertical stress induced from dynamic compaction as a function of depth and shear wave velocity, V_s , by assuming an average density, Poisson's Ratio and trapezoidal stress distribution beneath the centre of impact. Mayne and Jones (1983) were able to present good correlation between the measured stress and the actual stress, as presented in Figure 2-22.

Mayne and Jones (1983) postulated that as the density of the soil is increased by dynamic compaction, the peak decelerations and the maximum peak stresses will increase with increasing blows.



Figure 2-22. Comparison between Measured and Predicted Peak Stress (Mayne and Jones, 1983)

Jamiolkowski and Pasqualini (1992) presented a discussion paper regarding the quality control of dynamic compaction via Relative Density, D_R . They suggested that a better estimation of the densification of sands is obtained if the effects of the stress and strain history induced by the dynamic compaction process are considered. This was presented through the relationship between the cone resistance from a CPT, q_c , and the relative density, which is expressed as follows:

$$D_R = \frac{1}{2.38} \ln \left[\frac{q_c}{248(\sigma_{ho})^{0.55}} \right]$$
[2-11]

Jamiolkowski *et al.* (1992) noted that Equation [2-10] is less applicable in overconsolidated sands and soils improved by dynamic compaction because of the difficulty in assessing σ_{ho} ². They go on to note that dynamic compaction of granular soils generally generates effects similar to those produced by cyclic prestressing, and the effect on the cyclic stress and strain history must be considered when densification is carried out to avoid liquefaction. Leonards *et al.* (1980) presented data regarding the use of dynamic compaction to compact granular fill. They noted that there appears to be an upper limit to the degree of compaction or densification that can be achieved; but noted that that additional data is needed to verify this hypothesis. The data presented by Leonards *et al.* (1980) is shown in Figure 2-23.



Figure 2-23. Cone Penetration Resistance versus Normalised Energy (Leonards *et al.*, 1980)

The effect of dynamic compaction on loess soils was investigated by Hu *et al.* (2000) whereby the authors carried out laboratory compaction tests to establish a relationship between engineering properties of the soil and blow count. In addition, the change in microstructure of the loess soils was investigated at the initial stages of compaction and after repeated compaction (up to 32 blows).

The dynamic compaction testing was carried out dropping a 10kg hammer repeatedly from a height of 1000mm to compact the specimen in a mould. An illustration of the setup of the apparatus is shown in Figure 2-24.

After a predetermined number of blows with the hammer (1 - 8, 10, 13, 16, 20, 26, 32) samples were removed from the apparatus and subject to testing to assess engineering properties, and also the change in microstructure of the soil profile.



Figure 2-24. Laboratory Compaction Testing (Hu et al., 2000)

The results of the investigation by Hu *et al.* (2000) showed that the rate of (vertical) settlement was initially high and the rate of settlement decreased with increasing blow count. Hu *et al.* (2000) noted that the change in gradient of the vertical settlement versus blow count occurred around 8 blows, however this was material-specific.

The results of the laboratory testing presented by Hu *et al.* (2000) showed some interesting results, in particular the results of the undrained shear strength versus blow count for different confining pressures in consolidated undrained triaxial tests. As can be seen from the results presented in Figure 2-25, the shear strength decreases after reaching a peak value.



Figure 2-25. Shear Strength versus blow count of Loess (Hu et al., 2000)

It can also be seen from this figure that the shear strength is dependent upon the confining pressures; that is, the greater the confining pressure, the higher the peak shear strength. Hu *et al.* (2000) also noted that the number of blows at which the peak strength occurred was

approximately the same; hence, it was somewhat independent of the confining pressures. Hu *et al.* (2000) related the shear strength to blow count as follows:

$$\tau = \tau_0 + \frac{aN}{1 + \left(\frac{a}{b} - 2\right)\frac{N}{N_c} + \left(\frac{N}{N_c}\right)^2}$$
[2-12]

where τ = shear strength of the compacted soil (kPa);

 τ_0 = shear strength of undisturbed specimen (kPa);

N = number of blows;

 N_c = Number of blows corresponding to peak strength of soil.

The two parameters a and b are termed regression parameters and are presented graphically on Figure 2-25 on the plot where the confining pressure is 100kPa (for example). Based on the data presented, Hu *et al.* (2000) calculated the parameters a and b as 2.14 and 1.19, respectively.

The work Hu *et al.* (2000) carried out into the change in microstructure of the soil can be summarised as follows:

- The size of the aggregates increases near-linearly with blow count, reflecting an observed integration of soil grains or aggregates. Hu *et al.* (2000) also noted that the unstable grains are generally integrated in the initial blows, after which some of the aggregates further integrate and others disintegrate. Although these are opposite processes, the net result is a decrease in complexity of the particle distribution structure and, as a result, an increase in density.
- Under impact loads, the shape of the grain changes very slightly, but the orientation varies significantly.
- The surficial roughness of the particles and aggregates increases initially, which corresponds to the integration of soil grains and particles. With increasing blow count, the surficial roughness decreases, which is consistent with the integration processes noted above.

• The size and density of the pore spaces mirror those of the particles with blow count, which is expected as the alteration of the pore size and gradation is dependent on the deformation of the particles.

Hu *et al.* (2000) did note that for loess soils, dynamic compaction can be used to compact the soils to increase the density and reduce collapsibility, however over-compaction should be avoided. The design of the dynamic compaction process should be such that the number of blows coincides with achievement of the peak strength of the soil.

Further work on the use of digital image analysis to assess the behaviour of clay fills subject to dynamic compaction was presented by Hu *et al.* (2005). As part of their studies, the authors looked at the accumulated settlements with increasing number of blows from a falling hammer, which is shown in Figure 2-26.



Figure 2-26. Ground Settlement with Blow Number (from Hu et al. 2005)

As can be seen from the graph, settlement induced by dynamic compaction occurs in three stages, namely *Stage I* – the accumulated settlements occur quickly; *Stage II* – the accumulated settlements increase slowly; and, *Stage III* – where the accumulated settlements increase quickly.

The microstructural change of the clay soils was analysed by Hu *et al.* (2005) using software called Micro-structural Image Processing System (MIPS), which converts the digital image into a ternary image, from which parameters of the clay structure can be assessed. The key morphological parameters were the particle number, total particle area, percentage pore area, average shape factor, orientation and maximum chord length. The authors concluded that with increasing number of blows, the particle number, percentage pore area, average shape factor and orientation decreased. The percentage of the total particle area and the average maximum particle chord length increased with number of blows, which is consistent with settlement data.

2.3.3 Case Studies

Mayne *et al.* (1984) compiled field measurements from over 120 sites to determine if the response of the ground to dynamic compaction can be characterised. The ground response to dynamic compaction was categorised into five groups, namely: induced subsidence, ground vibrations, depth of influence, pressuremeter tests and penetration tests.

Mayne *et al.* (1984) presented results of the crater depth, both total depth and a normalised depth (with respect to the square root of energy per blow), with blow count. In addition, the total ground surface settlement was compiled and plotted versus applied energy in $t.m/m^2$. These plots are reproduced in Figures 2-27 to 2-29.



Figure 2-27. Crater Depth vs Blow Count (Mayne et al., 1984)



Figure 2-28. Normalised Depth vs Blow Count (Mayne et al., 1984)



Figure 2-29. Subsidence vs Energy (Mayne et al., 1984)

As can be seen from the above Figures 2-27 to 2-29, the depth of the crater and also the total net settlement typically increases with the energy used for dynamic compaction. This observation was also made with regard to the ground vibrations monitored at some of the

sites compiled by Mayne *et al.* (1984). They noted that the attenuation of the ground vibrations (or Peak Particle Velocity) is site-dependent and related to the scaled distance (the horizontal distance divided by the square root of the energy) via the drop height and weight of the pounder.

The depth of influence is a design input for most dynamic compaction projects, with the deepest recorded depth (to date) being at Nice Airport, where the densification by the dynamic compaction extended down to >33m (Mayne *et al.*, 1984) using a machine that delivered approximately 3,900 t.m of energy per blow. Presented in Figure 2-30 is a summary plot of the depth of influence against the square root of the weight of the pounder times drop height. From the results presented, it can be seen that the depth of influence can vary between 1/3 of \sqrt{WH} and as high as 1 times \sqrt{WH} .

Numerous other studies have been published regarding the application of dynamic compaction at different sites around the world. The studies often explain what was done, i.e. the dynamic compaction process implemented and the testing undertaken to assess post-dynamic compaction geotechnical properties of the fill.



Figure 2-30. Summary plot of Depth of Influence (Mayne et al, 1984)

Presented in Table 2-5 is a summary of the case studies reviewed as part of this Literature Review. A summary of the case studies presented in Table 2-5 and the collation of the 120 sites by Mayne *et al.* (1984) is as follows:

- Dynamic compaction has been carried out in a range of soils to a varying degree of success.
- Dynamic compaction is typically undertaken for highways (or roads); and for commercial developments.
- The typical mass on the pounder ranges between 15t to 20t with drop heights typically between 15m and 25m.
- The geotechnical tools for assessing the effect of dynamic compaction varies from project to project, however the majority of testing incorporated topographic survey to provide an overall response of the ground surface profile. Typically, the assessment tools consisted of SPTs, CPTs and/or pressuremeter tests. Of particular interest is only one paper reported the use of direct density testing (carried out in test pits) as part of the geotechnical testing strategy.
- The depth to which dynamic compaction compacts the ground is reasonably consistent and proportional to the square root of the pounder weight multiplied by the drop height (\sqrt{WH}). Where a significant amount of particle realignment occurs, such as in coralline soils, then the depth of influence presented by Mayne *et al.* (1984) is not applicable.

The above observations provide guidance for developing a basis for a trial of dynamic compaction. And in particular, highlights the need to identify a geotechnical investigation technique that accurately measures the required geotechnical parameters – and in the case of collapse settlement, the required geotechnical parameter is density.

Reference	Proposed	Description
	Development:	
Charles et al.	Industrial	The paper presented five case studies where dynamic compaction has been used to compact varying loose
(1981)	developments	fill.
	Hospital	The sites included:
	Roads / highway	• a restored open cast ironstone mining site in Corby;
		• an old refuse tip site on the outskirts of London;
		• an Old Domestic Refuse site in Redditch;
		• an old domestic refuse site in Hertfordshire; and,
		Soft alluvial soils in Guildford
		Dynamic compaction was applied by dropping a 15t weight from a 20m drop height. At Guildford two
		stages were carried out an interval of 10 days apart using a 20t pounder being dropped from 15m high.
		Typically between 10 and 15 blows were carried out at each drop location.
		Settlement monitoring was undertaken for 5 ¹ / ₂ years after completion of dynamic compaction at Corby.
		The areas subject to dynamic compaction had a mean surface settlement of 31mm, which was large when
		compared to areas that were subject to a 6m preload (settlements were 6mm); however was comparable to
		that of the fill material that had not been loaded.

Table 2-5. Summary of Dynamic Compaction Case Studies

Reference	Proposed	Description
	Development:	
		At an old tip site on the eastern end of London, the induced settlement was 580mm over a depth of 6.5m, which represented a strain of 9%.
		At the site in Redditch, dynamic compaction reduced the amount of settlement to 1/3 of its originally estimated value; however reduction in creep settlement was not achieved.
		At the Guildford site, pore pressure measurements were taken during the dynamic compaction process and
		for a period of two months afterwards. The pore pressure monitoring results are showed distinctive
		increases in pore pressure which dissipated over time. However even after 2 months after dynamic
		compaction the pore pressure were 2m higher than the phreatic surface.
		Charles et al (1981) indicated that the performance of a filled landform was governed by the self-weight of
		the fill, rather than the additional loading induced from the development.
Lutenegger	Shallow	Examples were provided whereby dynamic compaction was used to densify loess soils for a development
(1986)	foundations	comprising shallow foundations. Lutenegger (1986) concluded that the collapse potential had
		significantly reduced through a breakdown of the natural soil structure. This was determined by the
		collection of near-undisturbed soil samples and the excavation of test pits for density testing.

Reference	Proposed	Description
	Development:	
Liong (1991)	Industrial	Dynamic compaction was carried out in coralline soils with limited success. The effective depth of
	development	treatment was limited to between 5m and 7.5m when using a 15t pounder falling from a drop height of
		20m. The limited effectiveness of the dynamic compaction was assessed by the author as being due to the
		crushing of the coral fragments rather than the compaction of the soil.
Thomson	Industrial	Dynamic compaction was carried out of a rockfill site on Tsing Yi Island, Hong Kong, using a 15t pounder
(1991)	development	being dropped from a 20m height. Large scale load testing of the resultant landform was carried out, with
		settlements monitored. Thomson (1991) assessed that the total settlement and angular distortion of the
		resultant landform would be in the order of 20mm and 1/2500, respectively.
Varosio et al.	Roads / highway	Dynamic compaction was used to compact loose hydraulic fill with a fines content in excess of 50%. The
(1993)		authors concluded that dynamic compaction was a suitable means of treating the soil for intended design
		of pavement foundations on which heavy gantry cranes would operate.
Whetten	Commercial	Results of dynamic compaction monitoring were presented for the remediation of a supermarket site, with
(1993)	development	subsurface conditions consisting of uncontrolled fill. Boreholes, with SPTs, were carried out after the
	(supermarket)	dynamic compaction process to assess the effectiveness of the process. In addition, vibration monitoring
		was also carried out.
		Whetten (1993) noted an increase in the peak particle velocity recorded of the second pass, when
		compared to the first pass which was interpreted to be indicative of an increase in fill density between the

Reference	Proposed	Description
	Development:	
		first and second pass, however no definitive testing was reported.
Dumas et al.	Commercial	Dynamic compaction was used on a site comprising of approximately 2m of sandy material overlying 4m
(1994)	warehouse	of silt which in turn overly 20m of sandy silt to silty sand. The water table at the site was approximately 5m below the ground surface level.
		Dynamic compaction was carried out in 4 phases. The spacing adopted was 14m centre-to-centre, with subsequent passes set out on a grid at the centre of the preceding phase drop point. The first 3 phases were carried out using sand replacement techniques that lead to the formation of sand columns 2.5m to 3m in diameter, and extending to depths up to 6m. The average energy used to compact the ground was 27 t.m/m ³ of soil treated. Significant improvement in the elastic modulus was observed to a depth of 12m below the ground surface, with the most significant improvement occurring between depths ranging of 3.5m to 7.5m below the groundwater table.
Swedenborg	Bridge foundation	The results of CPT and pressuremeter testing were presented to show the effectiveness of dynamic
(1998)		compaction for bridge foundations between Sweden and Denmark. The works carried out indicated that the dynamic compaction process was adequate in compacting the subsurface profile (filled site) to generally achieve an elastic moduli sufficient for design requirements.

Reference	Proposed	Description
	Development:	
Wang <i>et al.</i> (2000)	Commercial warehouse	 Dynamic compaction was used in conjunction with vertical wick drains to consolidate soft clays. Vertical drains were installed across the trial area at 1.6m centres on a triangular spacing. The dynamic compaction was carried out on a 6m x 6m grid pattern with three passes of the dynamic compaction. For the first two passes, at each drop location a 15 tonne pounder was dropped 7 times from approximately 13m high. For the third pass, the same weight was used and a drop height of 10m was adopted with only 6 drops at each location. A final pass was subsequently carried out using two drops of the pounder from approximately 4m high. Testing carried out included plate load tests, CPTs, vane shear tests, piezometer monitoring, magnetic settlement and soil pressure cells. In summary, the results of the testing indicated the following: Pore pressure measurements were made, with the results showing that excess pore pressures dissipated within 7 days. Settlements monitoring indicated that rapid settlement occurred as a result of the dynamic compaction, which was approximately 50% of the total settlement monitored. The majority of this settlement occurred within the primary pass. The CPT results indicated an increase in cone resistance immediately after dynamic compaction and also 4 and 6 months after compaction, due to the dissipation of pore pressures and consolidation of the
		5011.

Reference	Proposed	Description
	Development:	
Al-Dafiry	Commercial	Dynamic compaction was used in Yemen to treat loose silty sand materials. Assessment of the dynamic
(2003)	development	compaction process was carried out using SPT N values, which increased between 2 and 5 times their original value.
Chun and	Industrial	A trial of ground treatment methods (one of which being dynamic compaction) was described for a
Yeoh (2003)	development	 municipal landfill that was to be used as a construction site in Korea. The depth of the landfill was of the order of 100m. It was concluded from the results from the trial that the optimum number of blows was 4 to 5 using compaction energies of 225 to 300 t.m and that the effective improvement depth was of the order of 11m. Based on an assessment of vibrations during the trial that the dynamic compaction should be at least "60m away from cultural properties, 40m from house or apartment, and 35m from shopping mall".
Miller et al.	Roads / highway	Dynamic compaction was carried out, with assessment being made using the flat plate dilatometer (DMT).
(2004)		The post-dynamic testing using the DMT was sensitive to the densification process through the assessment
		of the horizontal stress index values and the constrained modulus. Miller et al. (2004) also noted that the
		maximum improvement occurred at a depth of about 11 to 13 feet (3.5m to 4m).

CHAPTER 2

Reference	Proposed	Description
	Development:	
Miao <i>et al</i> .	Roads / highway	Dynamic compaction was carried out using two energy level, 150t.m and 250t.m.
(2006)		Monitoring of the pore pressures generated during dynamic compaction was carried out by. In addition
		the authors carried out CPT probes and spectral analysis of surface waves (SASW) to assess the
		effectiveness of the dynamic compaction process.
		The authors concluded that the pore pressure generated stabilised after approximately 6 blows for depths
		up to 7m and for an energy of 150tm. The stabilisation of excess pore pressures at a depth of 9m occurs
		after 1 blow, thus indicating the effective depth of treatment. For the case of an energy of 250tm, the
		shallower depth indicate a stabilisation of the pore pressures after 4 to 5 blows, with the pore pressures
		still increasing at a depth of 9m after 5m. Similar observations were also made by the authors for the
		monitoring that was carried out of the lateral movement of the soil with increasing blow count.
		The results of the CPT and SASW testing carried out by the authors were used to assess the degree of
		improvement and the effective depth of treatment, the results of which were used to develop an empirical
		relationship between the depth of treatment and the quasi-static pressure of the pounder.

2.4 Summary

Based on the literature review of the works that have been published to date, there has been a substantial amount of work carried out on both collapse settlement and on the analysis of dynamic compaction, based on both empirical results (field trials) and numerical modelling.

A number of areas of incomplete knowledge have been identified in the research that has been carried out to date, and these are categorised into the two parts of the thesis.

Part 1 – Understanding and Treating Fill

The relationships and case histories presented in the literature do not extend further to presenting quantified relationships between engineering properties (such as density or stiffness) to collapse potential. Authors such as Lawton *et al.* (1989) have noted relationships between collapse potential and density, but did not quantify the relationship. Charles and Watts (2001) and Charles (2003) also noted there was limited applicability of field and laboratory tests to identify collapse potential as these tests are usually reported in terms of overall strength and compressibility, not collapse strain in relation to the density of the soil.

The current literature has a significant limitation on the quantification of collapse strain in relation to density and also the development of a reliable means of assessing the *in situ* collapse potential of fill.

Further limitations also exist as to quantifying the impact of the collapse potential on the ground surface profile, and the development.

The objective of the first part of this thesis will be to present further research into identifying the mechanisms involved in the collapse of soil, with the focus then being on quantifying the magnitude of the collapse with respect to the density and moisture content of the soil.

A key limitation that is to be addressed is the development of a robust methodology for accurately assessing the *in situ* density of the fill, and hence the collapse potential. This is even noted in some of the studies undertaken for dynamic compaction where numerous authors (Chow *et al.*, 1992, for example) discuss the "densification" of dynamic compaction, but do not measure the density of the subsurface profile; rather they infer

density from either CPT or SPT results; or quantify densification through a parameter such as friction angle.

The final objective of Part 1 of this thesis is to use the methodology for assessing the *in situ* density of the fill after treatment with dynamic compaction; and to undertake an assessment of the resultant landform to quantify the likely performance resulting from collapse settlements.

Part 2 – Understanding Dynamic Compaction

The literature presented on the depth of improvement is primarily focused on the results from a single drop point; however the application of dynamic compaction is undertaken in an interlocking grid pattern with a varying number of blows. The relationship between the grid spacing, number of blows and depth of improvement has not been explored and is a shortcoming of the literature. Therefore, the energy requirements of grid spacing and number of blows have been the subject of a full scale trial where quantification of the effect of different number of blows; and different grid spacing has been considered.

The literature on the effects of dynamic compaction has solely been focused on the compaction or settlement resulting from dynamic compaction. Lukas (1992) did investigate some lateral deflections resulting from dynamic compaction; however none of the literature presents information about the near-surface disturbance, or heave, which results from dynamic compaction. Therefore, a key objective of the study of dynamic compaction is the effect that dynamic compaction has on the near surface, or Heave Zone.

Historical relationships have been presented for the depth and magnitude of settlement resulting from dynamic compaction. Based on the results of a full-scale trial, the previous empirical relationships are refined with amendments to previously established relationships provided to increase the knowledge in the dynamic compaction area.

In summary, the key objectives are as follows:

- 1. <u>Part 1 Understanding and Treating Fill</u>
- A. Identify the mechanisms involved in collapse settlements, and quantify the magnitude of the collapse with respect to the density and moisture content of the material.
- B. Develop a methodology for assessing accurately the *in situ* density of the

soil.

- C. Develop a methodology for assessing the effect on the ground surface profile, and buildings, of a collapsible zone occurring within the fill.
- 2. <u>Part 2 Understanding Dynamic Compaction</u>
- A. Develop methodologies for assessing the energy required for dynamic compaction, based on the results of a full-scale trial of dynamic compaction on a subsurface profile comprising uncontrolled fill. The energy requirements would be:
 - a. The grid spacing; and,
 - b. The number of blows at each drop location.
- B. Identify theory for estimating the effect of ground heave resulting from dynamic compaction, and calibrate the theory with the results of a full scale dynamic compaction trial.
- C. Review literature and provide further refinement of historic empirical relationships regarding the depth and magnitude of settlement induced by dynamic compaction based on the results of the full scale trial.
- D. Assess whether dynamic compaction has been successful in compacting the ground to reduce the impact of voids in the fill which possess the potential to collapse.

CHAPTER THREE

LABORATORY TESTING, GEOTECHNICAL TESTING AND CALIBRATION

3 LABORATORY TESTING, GEOTECHNICAL INVESTIGATIONS AND CALIBRATION

3.1 Introduction

In order to successfully apply any form of ground improvement, a thorough understanding of the ground conditions, including the nature of the material, is required. As discussed in Chapter 2, the settlement of deep fills has been the subject of significant studies, particularly associated with the subsequent performance of houses constructed over deep fills.

In the context of landform performance, there is a potential risk of unsatisfactory performance of residential buildings and associated infrastructure built over uncontrolled or poorly compacted fill. A number of settlement mechanisms have been identified that have implications for the expected landform performance. These include:

- Collapse settlement of the lower fill due to inundation;
- Settlement of the lower fill under load;
- Settlement of the fill below the water table under load; and,
- On-going secondary compression otherwise known as creep settlement.

Collapse settlement is a phenomenon that has been documented around the world as noted in Section 2.2. Houston *et al.* (2003) noted that collapse settlement occurs in recently deposited soils, such as alluvial or colluvial deposits in arid to semi-arid environments. The phenomenon has also been observed in man-made fills where placement has resulted in a loose or poorly compacted deposit (Houston *et al.* 2003).

The magnitude and rate of collapse settlement represent a significant issue when a development is proposed within these geological environments. The issue has the potential to be particularly significant in the case of residential and commercial developments, where the likelihood of collapse is linked to a range of potential water sources. These include infiltration from rainfall; groundwater level rise; leaks in water bearing services or pools and/or water basins, channels or lakes.

The issue of collapse settlement was identified as a potential issue for the filled sites intended to be developed for residential and commercial purposes because of the methods of fill placement and the potentially low density of the fill material.

The initial part of this chapter focuses on the characterisation of the fill material and the quantification of the collapse potential in relation to the density at which the material was placed.

Geotechnical investigations were undertaken within areas that were going to be treated (with dynamic compaction). An important element of the geotechnical investigation, apart from identifying the in situ conditions, was also to calibrate tools that could be used to correlate the expected behaviour derived from laboratory testing to the in situ conditions. Therefore, the second part of this chapter examines at the geotechnical investigations undertaken and links them to the results of the laboratory testing, thus allowing assessment of the resultant landform (i.e. after dynamic compaction) that is commensurate with the nature of the fill.

3.2 Laboratory Testing

3.2.1 Fill Material Properties

Fill material properties have been determined using a series of classification tests. The following classification tests were carried out:

• **Particle Size Distribution Tests:** Particle size distribution tests are carried out by sieving the soils through a "nest" of sieves of differing seizes in accordance with Australian Standard AS 1289.3.6.1. The percentage, by mass, retained within each sieve is calculated to provide a distribution of the grain size of the soil sample.

Plots of the particle size distribution tests can be seen on Figure 3-1 below.

The grain distribution is indicative of the way the soil sample will behave. Depending on the particle size distribution, and the results of Atterberg Limit testing, the soil can be classified in accordance with the Unified Soil Classification System (USCS).

• Atterberg Limits Tests: Atterberg Limits tests measure the moisture content at which the soil changes from the solid to the plastic state (termed the plastic limit) and from the plastic to the liquid state (liquid limit). The difference between the liquid limit and the plastic limit is called the Plasticity Index.

Atterberg Limit tests are carried out on the fine grained soils such as clays and silts (material finer than 425µm) in accordance with Australian Standard AS 1289.3.1.1.

Plots of Atterberg Limits are presented on Figure 3-2. The results of the Atterberg Limits tests are superimposed on Casagrande's plasticity chart. Depending on the Liquid Limit and the Plasticity Index, and the particle size distribution, the soil can be classified in accordance with the Unified Soil Classification System (USCS).

- Specific Gravity (Soil Particle Density) Tests: Specific Gravity tests involve using a hydrometer to calculate the density of the solid particles within the soil structure in accordance with ASTM D792-66(79). As the specific gravity is solely related just to the solid particle of the soil fabric, it is stress independent.
- Moisture Content Tests: Moisture content tests are carried out by weighing the soil sample in its current (or in situ) state, drying out the soil sample in an oven and then re-weighing the soil sample in accordance with Australian Standard AS 1289.2.1.1. The percentage change in mass allows the moisture content of the soil sample to be calculated.

Moisture content tests are typically used to compare the in situ moisture content to that of the Atterberg Limits (to assess the state that the soil is in) or to the results of Standard Compaction Tests (to assess a relative measure of how dry or wet the soil is).

• Standard Compaction Tests: Standard compaction tests are carried out to determine the maximum achievable density over a range of moisture conditions and are carried out in accordance with Australian Standard AS 1289.5.1.1. The test is usually undertaken by compaction of the soil in three layers in a standard size mould using a standard energy (2.7kg hammer, drop height of 300mm and 25 blows) to compact the soil the three layers approximately 40mm thick. The test is carried out over a range of moisture contents to determine the Standard Maximum Dry Density (SMDD), which occurs at the Standard Optimum Moisture Content (SOMC).

A total of 50 suites of the above tests were carried out on bulk soil samples collected from random locations around the entire site (refer to Figure 1-1), not just the dynamic compaction trial area. The samples were collected from various parts of the site in order to classify the broad range of materials likely to be encountered within the dynamic compaction trial areas.

Summary plots of the particle size distribution test results, the Atterberg Limit test results and a summary of the compaction test results are presented in Figures 3-1 to 3-3 respectively; with a summary table of the laboratory testing carried out in Appendix 3A.

The following key points pertaining to the laboratory testing:

- The particle size distribution test results (Figure 3-1) indicated that between 15% and 91% of the material was retained by the 75µm sieve. Only 4 of the samples had material retained in the 2.36mm sieve, indicating that there was limited gravel within the samples collected.
- The Atterberg Limits test results (Figure 3-2) indicate that those tests that contained plastic fines (i.e. material <75µm), were of low plasticity. Only one sample tested had a Liquid Limit greater than 35%, indicating it is of medium plasticity.
- Based on the results of the particle size distribution tests and the Atterberg Limit tests, the soil material can be classified as follows:
 - Approximately 50% of the samples are a Sandy Clay (CL) of low plasticity;
 - Approximately 30% of the samples are a Silty Sand (SM).
 - Approximately 20% of samples would be classified as a Sandy Silt (ML) of low plasticity, with only a few samples classified as a medium plasticity silt.
- The results of the Specific Gravity tests indicated a particle density of between 2.63 and 2.71 (with an average of 2.68). The average Specific Gravity has been used to develop the No Air Voids Line, shown on Figure 3-3.
- The compaction test results are plotted on Figure 3-3 and show that the Standard Maximum Dry Density ranged between 1.70t/m³ and 1.96t/m³; with an average of 1.85t/m³. Apart from one outlier the Standard Optimum Moisture Content ranged between 10% and 15%; with an average of 13%. These results are typical for low plasticity Sandy Clays and Silty Sands.
- Typically, the moisture content test results ranged between 3.7% and 15.2%, with an average moisture content of 8.3%. This equates to the moisture content of the samples tested ranging between 0.8% and 7.7% dry of the Standard Optimum Moisture Content, with an average of 4% dry of the Standard Optimum Moisture Content.



Figure 3-1. Summary of Particle Size Distribution Test Results



Figure 3-2. Summary of Atterberg Limit Test Results



Figure 3-3. Summary of Density Test Results

3.2.2 Laboratory Collapse Testing

Laboratory testing was carried out to assess collapse settlement of the soil and to assess if a relationship exists between the magnitude of collapse settlement, density and overburden pressure.

American Standard, ASTM D5333 - 92, was used for this purpose. The test method consists of placing a soil sample in a consolidometer, applying a predetermined applied vertical stress and then inundating the sample with fluid to induce collapse.

The methodology adopted for the laboratory collapse testing has been developed based on ASTM D5333 - 92, and consisted of the following:

- Soil samples were compacted into 75mm (diameter) by 150mm (height) moulds to dry density ratios of 90%, 95% and 100% of SMDD at 3% dry of the SOMC. The SMDD and SOMC were determined for the particular soil sample used in each test.
- Soil samples were then surcharged to a nominated stress of approximately 80kPa, 120kPa or 160kPa, with settlement under the surcharges measured over time. The nominated stress levels typically coincide with a total stress level at depths of 4m, 6m and 8m.

• Once settlement under surcharge reached a negligible rate, the samples were saturated by simply flooding the contained with water. Settlement during and after saturation was also measured over time. The collapse strain is calculated as the change in height (settlement) of the sample over the height of the sample immediately prior to saturation of the sample.

A sketch of the testing apparatus for assessment of collapse potential is shown in Figure 3-4.



Figure 3-4. Sketch of Laboratory Test Apparatus

A total of 50 collapse settlement tests were carried out for varying density ratios and surcharges on a silty sand and silty clay. The particle size distribution for the silty sand and silty clay is shown on Figure 3-5.


Figure 3-5. Particle Size Distribution

A summary table presenting the number of samples tested for the different surcharge and density ratio for silty sand and silty clay materials are presented in Tables 3-1 and 3-2, respectively.

Table 3-1.	Summary	of Silty	Sand	Testing
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	Number of Tests Carried Out			
	80kPa Surcharge	120kPa Surcharge	160kPa Surcharge	
90% SMDD	1	2	1	
95% SMDD	1	1	1	
100% SMDD	1	2	1	

The results of the laboratory testing are presented as a plot of settlement versus time. A typical example of the results of the laboratory testing is presented in Figure 3-6. The example provided is for a silty clay material, compacted to density levels between 90% SMDD and 100% SMDD and under a 160kPa surcharge.

	Number of Tests Carried Out			
	80kPa Surcharge	120kPa Surcharge	160kPa Surcharge	
90% SMDD	4	5	4	
95% SMDD	4	6	4	
100% SMDD	4	4	4	

Table 3-2.	Summary	of Silty	Clay	Testing
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Inspection of Figure 3-6 indicates that the results can be subdivided into two distinct stages, namely before saturation and after saturation. An example of the closer examination of "before saturation" and "after saturation" stages is presented in Figures 3-7 and 3-8, respectively. It should be noted that the time axis in Figure 3-8 is the time after saturation.



Figure 3-6. Example of Laboratory Collapse Settlement Test



Figure 3-8. Results of Monitoring After Saturation

The values of collapse strain calculated from laboratory collapse testing results are presented graphically as a function of the applied surcharge and the material type. Figures 3-9 and 3-10 present this information for the silty sand and silty clay materials, respectively. Also presented on the figures is an "upper bound" trend line of collapse strain for each of the overburden pressures applied during testing.

The results of the laboratory testing program can be summarised as follows:

- The collapse strain is a function of the overburden pressure and the dry density ratio at which the material is placed.
- The amount of initial settlement that occurs is dependent on the compaction level, surcharge and material type.
- After initial settlement upon application of vertical stress, the soil structure tends to remain stable until inundation, after which settlement occurs rapidly.
- The material type has a significant influence on the magnitude of collapse settlement, with sandy materials indicating less collapse settlement than clayey material.

There is some scatter in the results presented in Figures 3-9 and 3-10 which can be attributed to some variability in the material used in the tests. While the material can be classified as a "silty sand" or a "silty clay" there is some variability in the clay content of the material which would contribute to the scatter (refer to Section 2.2 of Chapter 2, for further discussion about collapse settlement).



Figure 3-9. Silty Sand Material



Figure 3-10. Silty Clay Material

The results of the laboratory testing presented above are comparable to those presented in literature. Charles *et al.* (1979) presented the results of full scale inundation trials where 100mm of settlement was induced in the upper 7m. This equates to a collapse strain of about 1.4%; which is comparable to the results presented in Figure 3-10 for a low overburden pressure.

Charles and Watts (2003) noted that "ground movements due to collapse compressions may be large, and vertical compressions of the order of 3% to 5% are not unusual." The general range presented by Charles and Watts (2001) and the range in settlements noted in Chapter 2 are generally consistent with the results of the laboratory testing presented herein.

Furthermore, Charles and Watts (2003) noted that "the results of the BRE test programme suggest that 95% relative compaction based on the standard Proctor Compaction test might largely eliminate collapse potential with some coarse fills, but with fine fills there could be a collapse potential of as much as 2% where the fills are placed dry of optimum moisture content." The data presented in Figures 3-6 and 3-7 is consistent with the statement made by Charles and Watts (2003); whereby for the silty sand sample had collapse settlement <0.25% when compacted to 95% of the SMDD; however the silty clay sample had potentially up to 1% collapse settlement at the same density relationship.

The data presented in Figures 3-9 and 3-10 could be used to estimate the collapse strain at various depths within the soil profile if sufficient data regarding achieved dry density ratio were available. It should be noted that the above results were achieved at a moisture content approximately 3% dry of SOMC. Different results would be expected with different initial soil moisture contents – that is, with increased moisture content there should be a decrease in collapse potential.

Observation of the plots indicates that there is a significant difference between collapse strain recorded for the silty sand material and the silty clay material. For example, at a density ratio of 92% and a surcharge of 160kPa, the collapse strain for the silty clay material is >4%, whereas for the silty sand, the collapse strain is <1.5%.

It is noted that significantly fewer silty sand samples were tested and hence the relationship between collapse strain and dry density ratio is less developed than for the silty clay material. Notwithstanding the limited data for the sandy soils, it seems evident that the sandy samples in general have a lower potential for collapse settlement than the clay samples. Further discussion on the possible reasons for this observation is provided in later sections.

3.2.3 Electron Microscopy

3.2.3.1 Introduction

The nature of the collapse behaviour, and its relationship to inundation, begins at the fundamental level of the soil microstructure or fabric. Electron microscopy was identified as a means of examining the soil fabric and the nature of intergranular and interaggregate structures.

From the literature review presented in Chapter 2 (Section 2.6), it was evident that electron microscopy allows a qualitative assessment of the change in the soil structure before and after saturation. This way the characterisation of the soil structure can be undertaken, thus enabling assessment of the mechanism of collapse for these particular soil types.

The soil samples investigated as part of the electron microscopy study consisted of 2 samples of silty clay material and 2 samples of silty sand material. The sample preparation and the results of the testing are discussed in the following sections.

It should be noted that terminology specific to this thesis has been used to define certain aspects or characteristics of the soil structure, which includes:

- <u>Aggregations</u> Aggregations are large agglomerates of a sand grain plus silts and clays;
- Particles Particles are individual sand grains of various sizes;
- <u>Contacts</u> Contacts are the specific location where aggregations and / or particles are in contact.

Photographs were taken of the samples under two different magnifications, referred to herein as 100x and 1000x magnification. While the actual magnification of the image varies with the size of the presented picture, a scale is presented on each photograph for reference.

3.2.3.2 Sample Preparation

The preparation of the soil samples prior to being examined in the electron microscope involved each sample being placed in a mould, with holes drilled into the sides to allow saturation of the sample, and compacted to a density of 90% of the SMDD, at 3% dry of the Standard OMC. A compaction level of 90% SMDD was selected because it was the lowest value used in the collapse tests and it was expected to show the largest change in void structure. The samples were then surcharged with 120kPa, to reflect the *in situ* conditions of fill material located approximately 6m to 7m deep.

Saturation and monitoring of one of each of the silty sand and silty clay samples was carried out in a manner similar to that described above for the Laboratory Collapse Testing. Once settlement upon inundation (collapse settlement) had reached a negligible rate, all four samples were then prepared for examination in the electron microscope. The sample preparation involved sample extraction from the moulds and the breaking of the sample to produce a face that was as close to undisturbed as practicable. This face was then examined under the electron microscope.

3.2.3.3 Silty Sand Material

Photographs of the silty sand material prior to saturation under a magnification of 100x and 1000x can be seen in Figures 3-11 and 3-12.

Inspection of Figures 3-11(a) and 3-12(a) indicates that voids can be seen at both a 100x magnification and at a 1000x magnification between particles and aggregations of silty / clayey material. Small voids can also be seen surrounding larger particles, as observed under a 1000x magnification. Also noted under 1000x magnification is the discrete contact, with limited clay binding, between particles.



(a) Unsaturated

(b) Saturated





(a) Unsaturated





Photographs of the silty sand material after inundation are presented on Figure 3-11(b) and 3-12(b) at a magnification of 100x and 1000x.

Figures 3-11(b) and 3-12(b) show that after inundation there is a significant amount of clayey and silty colloidal material in between the aggregations filling the voids. Some voids still remain after inundation, particularly where particle-to-particle contacts form a stable structure.

Under 1000x magnification, it can be seen that before saturation the particle-to-particle contact is discrete with limited clay binding (refer to Figures 3-11(a) and 3-12(a)). After inundation, each of the particle-to-particle contacts, particle-to-aggregation contacts and aggregation-to-aggregation contacts are evident, with some infilling with colloidal material between the particle-to-particle and particle-to-aggregate structures.

Comparison of these observations indicates that where there are particle-to-particle contacts the soil structure is maintained even after inundation. Where there are particle-to-aggregation contacts the inundation process causes a weakening of the binding and collapse occurs with some infilling of colloidal material. In general terms, it appears the particle-to-particle configuration is a stable structure, whereas the particle-to-aggregate configuration is a metastable structure. The overall collapse behaviour in the silty sand material is therefore dominated by the weaker particle-to-aggregate bonds with a minor contribution due to the loss of soil suction between the particles.

These observations are consistent with Mitchell (1976) who noted that the following are required for collapse to occur:

- 1. An open, partly unstable, partly saturated fabric.
- 2. A high enough total stress that the structure is metastable.
- 3. A strong enough clay binder or other cementing agent to stabilize the structure when dry.

Mitchell (1976) stated that "when water is added to a collapsing soil in which the silt and sand grains are stabilized by clay coatings or buttresses, the effective stress in the clay is reduced, the clay swells, becomes weaker and contacts fail in shear. Thus compatibility with the principle of effective stress is maintained on a microscale."

It is noted that the soil structure does not show signs of cementation, either before or after inundation. This is evident from examination of the form of the contacts between the aggregations and the particles, which shows discrete points of contact.

Silty Clay Material

Photographs were taken of the silty clay material under 100x and 1000x magnification and are presented on Figure 3-13(a) and 3-14(a). Inspection of Figures 3-13(a) and 3-14(a) indicates that a significant number of large voids can be seen within the silty clay matrix at both the 100x and 1000x magnification.

A comparison between the silty clay material (presented in Figures 3-13(a) and 3-14(a)) and the silty sand material (presented in Figures 3-11(a) and 3-12(a)) indicates that the materials have a different structure to the soil fabric. Within the silty clay material, there are larger aggregates, and more aggregation-to-aggregation and aggregation-to-particle contacts compared to the silty sand material.

Upon saturation, there is a significant decrease in the incidence of large voids of the silty clay material. Figures 3-13(b) and 3-14(b) indicate that the majority of the large voids observed in the pre-saturation sample have been infilled as a result of filling by colloidal material or via realignment of the soil structure. Some minor voids are still observed, however these are located adjacent to larger sand grains. This is consistent with observations and subsequent assessment of the collapse in the silty sand material. It should be noted that the void ratio of the samples before and after saturation has not been quantified as part of this study, hence is a shortcoming of the research. The description of the voids in the soil structure is based on observation only.

The aggregate-to-aggregate binding appears to have been softened, resulting in a collapse of the structure and filling of the voids, which is also the case for an aggregate-to-particle binding. This is consistent with the behaviour noted by Lawton *et al.* (1992) and Mitchell (1976).

3.2.4 Summary of Laboratory Testing

An investigation has been carried out into the settlement behaviour of soil-based fill materials. Where fill material has been placed at a low density, settlement also occurs due to collapse settlement, or strain, in addition to elastic and secondary settlements.

A relationship has been developed between collapse strain, dry density ratio and overburden pressure for two soil types. The testing indicated that the collapse strain was lower at all values of dry density ratio and overburden pressure for the silty sand material, compared to the silty clay material. All samples were prepared at a moisture content 3% dry of the SOMC which was close to the average moisture content of the in situ materials.

A shortcoming of the research to date has been that the samples have only been tested at a single moisture content (i.e. 3% dry of SOMC). Based on the available literature it is anticipated that with increased moisture content, the collapse settlement would also decrease; however this relationship has not been quantified.



(a) Unsaturated







(a) Unsaturated

(b) Saturated

Figure 3-14. Silty Clay – 1000x Magnification

The relationship suggests that for the silty sand material compacted to approximately 90% of the Standard Maximum Dry Density, the collapse strain could be up to 2% under a surcharge of 160kPa. As the density ratio of the material increases to above 95% of the SMDD, the collapse strain reduces to be less than 0.25%. For the silty clay material under the same conditions, the collapse strain could be greater than 4% if the material was compacted to approximately 90% of the SMDD. As the dry density of the silty clay material increases, the collapse strain also decreases to less than 0.5% at a dry density ratio of 98% of the SMDD and under a surcharge of 160kPa.

Investigation into the mechanisms of collapse settlement of the silty sand material using an electron microscope indicated that there was no cementation between particles and that the mechanism involved in collapse was a softening of particle-to-aggregate contacts. The

particle-to-particle contact observed before inundation appears to remain intact during the inundation process. There is a small increase in the contact area as result of inundation, however this is assessed as being colloidal material. The change in collapse appears to be dominated by the softening of particle-to-aggregate contacts with a minor contribution to the collapse settlement due to the loss of soil suction between the particle-to-particle bonds.

As for the silty sand material, observation of the silty clay material under the electron microscope both before and after inundation indicated a lack of cementation. The particle-to-aggregate and aggregate-to-aggregate contact, in the silty clay material, before inundation also appears to be discrete, however after inundation there was no indication of these contacts existing in discrete form. It is therefore concluded that the binding of the aggregate contacts appears to have softened and collapse of the structure occurred. It is likely that some colloidal materials also fill the void space during the inundation process, as was seen around the more stable particle contacts.

As noted in Section 2.2.2 (Chapter 2), Camapum de Carvalho *et al.* (1998) investigated the soil structure and the resultant collapse. Their investigation into a porous clay showed that the microstructure of a porous clay consisted of micro aggregates of particles held together by weak "clay bridges", resulting in a metastable and highly porous structure. These aggregations of particles have also been described by Lawton *et al.* (1992).

Both Camapum de Carvalho, *et al.* (1998) and Lawton *et al.* (1992) observed that for low clay content soils (<10% clay content), the clay acts as a binder between the silt and sand particles also forming a metastable structure. A metastable bonding can also exist as a result of the capillary suction in a partially saturated condition (Mitchell, 1976). Camapum de Carvalho *et al.* (1998) observations of soil structure indicated the potential collapse mechanisms in clay as being the softening of the clay binding at aggregate contacts and, in sands, the collapse mechanism is the loss of capillary suction.

The results presented herein are consistent with that of Camapum de Carvalho *et al.* (1998) and Lawton *et al.* (1992) in that it appears that the aggregates (clays and silts) form a metastable structure and that there is a weakening of the aggregate contacts.

Dudley (1970) noted, in particular, that the major components of collapsing soils are "*materials of bulky shapes such as occurs in silts, sand and gravels*". While this may be the case for naturally forming soil deposits, it is also applicable to filled sites, whereby

under poor compaction control, clay materials are able to form relatively "bulky" aggregates. The magnitude of the collapse associated with the aggregates is larger than that observed in sandy materials; however collapse is still prevalent in silty sand materials.

The extent to which the formation of aggregates within the soil structure occurs is dependent upon the distribution of fines within the sample. A limitation of the work undertaken by Dudley (1970) is that the quantification (and distribution) of fines within the soil samples was not undertaken, hence the relationship between the amount of collapse and the silt and clay content could not be presented.

3.3 Geotechnical Investigations

3.3.1 Site Selection

Geotechnical investigations were undertaken at the sites of the full scale dynamic compaction trial. The geotechnical investigations were carried out in order to assess the initial *in situ* engineering properties of the fill.

Nine different areas were investigated as part of the dynamic compaction trial, denoted as Areas 1 to Area 9, each approximately 1 hectare in plan area. The layout of the sites in the second phase of the trial is shown on Figure 3-15.



Figure 3-15. Layout of the Sites

Area 1 was constructed using scrapers, whereas Areas 2 and 3 were constructed using truck dumping methods with fill comprising layers of loose sandy material, interbedded with clayey silt material and gravels. The construction method used to form Areas 4 to 9 is unknown and hence part of the geotechnical investigation was to understand the formation of the landform at these Areas and hence whether collapse settlements would be likely.

3.3.2 Geotechnical Investigation

In undertaking the geotechnical investigations a range of investigation methods were used. The investigation methods were:

- Electric Friction Cone Penetrometer tests (CPTs);
- Augered borehole with Standard Penetration Tests;
- Flat Plate Dilatometer Tests (DMTs);
- In situ density tests; and
- Large scale test pits.

Within this section a brief discussion of the methods is provided.

Electric Friction Cone Penetrometer Tests

An Electric Friction Cone Penetrometer Test (CPT) is a common geotechnical investigation tool that involves pushing an instrumented cylindrical penetrometer, with a conical tip, into the ground at a rate of 20cm/s. Sensors in the cone record the resistance on the tip of the cone (referred to as 'cone resistance') and also the friction along a 135mm section of the sleeve of the cone (referred to as 'skin friction').

A cross section showing a typical section through a CPT probe is shown on Figure 3-16.



Figure 3-16. CPT Probe (from Lankelma)

The results of the CPT are a plot of cone resistance and skin friction with depth. The cone resistance and skin friction are used to infer geotechnical properties of the soil such as stratification, soil type and strength parameters.

The CPT is designed to be used in soils (sands and clays). As such it is susceptible to damage if gravels or rocks are intercepted during the pushing of the probe into the ground.

Augered Borehole with Standard Penetration Tests

Augered boreholes involve using a drilling rig to advance a borehole using solid flight augers. An engineering log of the subsurface profile, based on visual classification of the soil, was prepared for each borehole.

The drilling of the boreholes is undertaken through a variety of materials. The drilling method adopted for advancing the boreholes is dependent on the anticipated materials encountered on site, the targeted depth of drilling and the groundwater conditions. For the ground conditions (and groundwater conditions) anticipated at the site augured boreholes were considered appropriate.

During the drilling of the boreholes, Standard Penetration Tests (SPTs) were carried out. SPTs involve counting the number of blows with a 40kg hammer required to insert a standard split spoon sampler a total of 450mm. The results of the SPT are recorded at 150mm intervals, and are typically presented as an 'N value', which is the total number of blows required to penetrate the last 300mm. The N Value is used to assess the *in situ* engineering properties, based on empirical relationships.

The SPT has been carried out over many decades throughout Australia and worldwide and hence there are numerous correlations between the SPT N value and engineering properties. There is, however, limited correlation between the SPT N value and the *in situ* density of the soil.

The SPT does provide a sample of the soil, which is an advantage over any of the other testing techniques discussed herein.

Flat Plate Dilatometer Tests

The Flat Plate Dilatometer test (DMT) involves advancing a blade, typically 95mm wide by 220mm long, into the undisturbed ground. On the face of the blade is a 60mm diameter circular membrane that is pushed 1mm into the soil. The pressure required to push the membrane into the soil is recorded and from this the stress condition and elastic modulus can be calculated.

Photographs of the DMT are presented on Figure 3-17.





Figure 3-17. Photographs of the DMT

The DMT is a robust tool that can be advanced into and through thin bands of gravelly soils. Testing was typically undertaken at 300mm depth intervals so that a reasonable profile of the stiffness could be achieved with depth.

In Situ Density Testing

Three methodologies were considered for carrying out in situ density testing, namely using nuclear density gauges, taking tube samples during the drilling of boreholes and carrying out Downhole Gamma Density Testing.

These methodologies are discussed in the following subsections and a comparison between the different methodologies is discussed in Section 3.4.

Nuclear Density Gauge

The nuclear density gauge is a tool for measuring the density of soil that is commonly used as part of an earthworks quality assurance program. The testing consists of a radiation source that is inserted into the ground, typically up to 300mm. The radiation source emits a beam that is directed towards a receiving sensor (detector) that is located at the ground surface.

The detector counts the radiation particles that pass through the soil between the source and the receiving sensor. By knowing the difference between the radiation at the source and the detector the nuclear density gauge can be used to calculate the density of the soil. The methodology for the nuclear density testing is defined in Australian Standard AS 1289. 5.8.1 (field tests) and AS 1289.5.1.1, AS 1289.2.1.1 and AS 1289.5.4.1 (laboratory tests).

Tube Samples

A testing methodology was developed whereby an undisturbed sample would be taken of the soil during drilling that would allow assessment of the *in situ* wet density. Photographs of the sampling methodology are presented on Figure 3–18, and involved the following:

- Inserting a thin walled sampling tube at the base of a borehole to obtain a sample of the soil, collected within an inner plastic sampling tube.
- The sample and sampling tube were then cut to length to produce a section approximately 200mm long to reduce impacts from disturbance of the end of the tube.
- The tube and sample were wrapped in plastic to maintain the moisture condition, prior to returning the sample back to the laboratory.
- At the laboratory, the length of tube was accurately measured, as well as the total mass of the tube and sample. The sample was then extruded and the tube re-weighed to determine the mass of the soil. The internal diameter of the tube was measured.
- This allowed the wet density to be calculated.
- The extruded sample could then be visually classified and placed in the drying ovens to determine the *in situ* moisture content. By knowing the moisture content and the wet density, the dry density could also be calculated.

Downhole Gamma Density Testing

Downhole Gamma Density (DHGD) testing consists of lowering a probe, approximately 3.8m long, down to the base of a borehole, then raising it to the surface progressively while recording the natural gamma radiation, back-scatter radiation (or density) sidewall profile of the borehole in 10mm depth intervals.

Photographs of the down-hole gamma density meter are presented on Figure 3-19.



Figure 3-18. Photographs of In situ Density Testing

Natural Gamma rays are electro-magnetic waves emitted during the decay of radioisotopes (potassium-40) that occur in all soils and rocks. The DHGD tool incorporates a detector that measures the gamma rays from a volume of the surrounding soil. The amount recorded is dependent on the borehole diameter (or volume of soil around the detector) and the lithology (the soil profile, e.g. sand, clays, silts etc.) of the side wall. The natural gamma radiation is recorded in American Petroleum Institute units, or API, which is essentially a unit of radioactivity.



Figure 3-19. Photographs of Downhole Gamma Density Meter

The DHGD probe also contains a single detector, focussed density system using a 125-mCi Caesium 137 source to record the apparent wet density of the formation. The wet density is assessed based on a back-scatter approach. That is, rays are emitted from the Caesium source in the probe and the rays are either absorbed or reflected depending on the density characteristics of the material. A receiver on the DHGD tool records the number of rays that are reflected from the soil, thus assessing the wet density of the soil. The wet density is recorded in grams per cubic centimetre, or tonnes per cubic metre (t/m^3) .

The results of the DHGD testing are typically presented in the form of three plots. The first plot is that of the Natural Gamma in API; the second is the wet density in tonnes per cubic metre (t/m^3) ; and the third is the recorded diameter of the borehole, in centimetres. The borehole diameter is assessed by recording the extension of a calliper that presses the DHGD tool against the sidewall of the borehole. An example of the output of the DHGD testing is presented in Figure 3-20. The vertical offset on the plots (i.e. the different depths at which the natural gamma, gamma density and caliper width start recording) is attributed to the physical location of each sensor on the probe.



Figure 3-20. Example of Output from DHGD Testing

Large-Scale Test Pits

Large scale test pits allow for a direct visual observation of the subsurface profile to be made and bulk samples of the soil profile to be taken. Engineering logs are recorded of the subsurface profile encountered during the excavation of the test pits.

During the excavation of the test pits, nuclear density tests were carried out, at approximately 300mm depth intervals, which allowed assessment of the density profile at the completion of the dynamic compaction process.

A photograph of a large scale test pit is shown on Figure 3-21.



Figure 3-21. Photograph of Large Scale Test Pit

Geotechnical Investigations Undertaken

A summary of the tests carried out at each investigation station is presented in Table 3-3 and discussion of the results is presented in Section 3.3.3.

Site	CPT	Borehole with SPTs	DMT	Down-hole Gamma	Test Pit with Nuclear
				Density Tests	Density tests
Area 4	1	1 Borehole (8 SPTs)	-	1 test	1 test pit (20 Tests)
Area 5	1	1 Borehole (7 SPTs)	-	1 test	-
Area 6	1	1 Borehole (8 SPTs)	36	-	-
Area 7	1	1 Borehole (7 SPTs)	24	-	1 test pit (16 Tests)
Area 8	1	1 Borehole (6 SPTs)	15	2 tests	-
Area 9	1	1 Borehole (7 SPTs)	23	2 tests	1 test pit (19 Tests)

Note that as the method of construction was known in Areas 1 to Area 3, no testing was carried out.

3.3.3 Results of Geotechnical Investigation

The results of the geotechnical investigations are discussed in the following subsections. At the end of Section 3.3.3 a summary of the results is provided.

Electric Friction Cone Penetrometer Test Results

CPTs were carried out in Areas 4 to 9 by a specialist contractor.

Details of the CPT probes are provided in Section 3.3.2, and a typical example of the results of carrying out a CPT is presented on Figure 3-19. The results are presented in the form of plotting the tip resistance, q_c , and skin friction, f_s , which is a direct reading from the test. Plotting of the result generally involves presenting the tip resistance and skin friction on different scales to allow detailed review of the high range (>30MPa) and low range (<10MPa) of the results. Also included in the output is the Friction Ratio, F_r , which is defined by Lunne *et al.* (1997) as follows:

$$Fr = \frac{f_s}{q_t - \sigma_{vo}}$$
[3-1]

where q_t = cone resistance corrected for pore pressure effects;

 σ_{vo} = total stress

The friction ratio and the cone resistance can then be used to assess the type of soil encountered. A graphic depiction of the stratigraphy can also be seen on Figure 3-22. The assessment of the soil encountered is carried out in accordance to Figure 5.7 of Lunne *et al.* (1997), which is after Robertson *et al.* (1986).

Based on results of the CPTs, the following can be summarised:

- The CPTs extended to depths varying between 9.2m and 13.8m below the ground surface level.
- The subsurface profile generally comprised interbedded silty clays and silty sands of varying stiffness.



Figure 3-22. Example Output of CPT

The CPT involves a relatively sensitive testing technique; hence the CPT data should be viewed in conjunction with borehole information. Encountering cobbles or boulders at depth may give a "false" interpretation of the depth of shale. As an example, refusal was obtained with the CPT probe at 9.5m whereas the borehole drilled within 1m of the CPT probe, intersected refusal on shale at a depth of 13.5m. An example showing the comparison between the borehole log and CPT output is shown on Figure 3-23.



Figure 3-23. Engineering Log and CPT Output

The CPT does provide a near-continuous profile which is considered to be an accurate profile of the *in situ* conditions. Furthermore, a number of engineering properties can be inferred from the CPT results, including density index of granular materials (<u>not</u> the dry density ratio or bulk, or wet, density) and an elastic modulus.

Lunne *et al.* (1997) provides relationships between the results of the CPT probes and the elastic modulus, noting that for drained materials the relationship "*mainly depends on relative density, over-consolidation ratio and the mean stress level of the soil*". Considering the stress range of the tests undertaken, the relationships provided by Lunne *et al.* (1997) can be simplified to the following:

 $E = \alpha . q_c$ [3-2]

where α = factor ranging between 2 and 5 depending on the soil type.

Testing has been carried out as part of the trial works to assess the value of α in Equation 3-2 that is appropriate for the specific *in situ* conditions. Presented on Figure 3-24 are the CPT cone resistances and the elastic moduli inferred from the flat plate dilatometer testing. Also presented on Figure 3-24 are the lines indicating the range of α values (from 2 to 5). Comparison between the dilatometer modulus (referred to as E_{DMT}) and the q_c value indicates that there is a line of best fit where $E = 4.1q_c$ (with a Coefficient of Determination of 0.0282, which suggests a very weak correlation).

As can be seen from Figure 3-24, there is a large amount of scatter in the data, resulting in a low coefficient of determination. The large scatter in the data can be attributed to soil variability (in material type), and differences in test locations and depths. The elastic moduli inferred from the DMT measures the horizontal stiffness; whereas the qc measures the stiffness in the vertical direction. Hence, the large scatter in the data could be indicative of anisotropy of the material.

It should be noted that the research presented herein is not sensitive to the elastic modulus considered adopted during the analyses (refer to Chapter 4) and as such the relationship is considered reasonable.



Figure 3-24. Comparison between Dilatometer Modulus and CPT

Augered Borehole with Standard Penetration Tests

Boreholes were drilled at each investigation station with SPTs carried out in the borehole. The boreholes were drilled to depths between 9.4m and 13.4m and engineering logs were recorded of the ground profile intersection.

At completion of the logging of the borehole in Areas 4 to 9, the borehole was left open to allow the DHGD tests to be carried out.

It should be noted that a standing groundwater level was not intersected during the drilling of the boreholes. In some of the boreholes, water inflow was recorded however this appeared to occur in isolated boreholes and at discrete depths between 5m and 10m.

The results of the SPTs have been used to provide a "benchmark" for comparison with other tests. While the accuracy of the SPT is generally not as good as the other tests, it does provide a good tool to allow comparison across different testing techniques.

Flat Plate Dilatometer Tests

Flat Plate Dilatometer Tests (DMTs) were carried out at investigation stations in Areas 8, 9, 10 and 11. The results of the DMTs are presented as a "dilatometer modulus" E_{DMT} which is a lateral elastic modulus based on a fixed displacement (inherent in the test method).

In Situ Density Tests

A range of in situ density tests have been undertaken during the investigation of the sites in order to characterise the materials. The results of the density testing are discussed in the following sections.

Nuclear Density Gauge

Nuclear density gauge testing was undertaken in the large scale test pits and served two purposes, namely: to calibrate the other density testing techniques and to establish a correlation between the in situ wet density and the dry density ratio.

The in situ wet density is calculated by the nuclear density gauge test (as well as in the tube samples and downhole gamma density tool) by a direct measurement. Bulk samples were taken of the material subject to nuclear density gauge tests, from which the moisture content and laboratory compaction tests are carried out.

Details of the moisture content test is provided in Section 3.2.2; and, once the moisture content is known the in situ dry density can be calculated using the following equation (from Whitlow, 1995):

$$\rho_d = \frac{\rho_w}{1 + mc} \tag{3-3}$$

where $\rho_d = dry density (t/m^3)$

 $\rho_{\rm w} =$ in situ wet density (t/m³)

mc = moisture content, calculated from laboratory testing

The laboratory compaction test (refer to Section 3.2.2 for additional details) is used to derive the maximum dry density of the soil under a standard compactive effort (SMDD). Once the maximum dry density is known, the ratio of the dry density to the maximum dry density can be calculated for that specific sample.

The relationship between the dry density ratio and the wet density of the in situ material is sensitive to the moisture content and the nature of the material. For the site, sufficient data have been collected to allow a statistical relationship between wet density and the dry density ratio, as can be seen from Figure 3-25, for both the silty sand and silty clay materials.

Given in Table 3-4 is a summary of the density test results presented over a range of wet density from $1.5t/m^3$ to $2.5t/m^3$ grouped into $0.1t/m^3$ bands. Presented in Table 3-4 is the average dry density ratio, the lower quartile value and the standard deviation. Similarly, presented in Tables 3-5 and 3-6 is a summary of the silty sand and silty clay materials.

Wet Density		Dry Do	ensity Ratio	
(t/m^3)	Count	Average	Standard	Lower
			Deviation	Quartile
1.5 to 1.6	7	89.2%	3.3%	87.0%
1.6 to 1.7	15	87.7%	2.55%	86.5%
1.7 to 1.8	14	90.8%	4.8%	87.0%
1.8 to 1.9	31	91.3%	4.0%	88.0%
1.9 to 2.0	53	94.1%	3.5%	91.5%
2.0 to 2.1	94	97.0%	3.2%	95.0%
2.1 to 2.2	58	100.2%	2.2%	98.5%
2.2 to 2.3	27	103.8%	1.9%	102.5%
2.3 to 2.4	1	112.5%	-	112.5%
2.4 to 2.5	1	102.0%	-	102.0%

Table 3-4. Summary of Density Testing – All Samples

Based on the results of the laboratory collapse settlement data, materials with a dry density of less than 95% of the SMDD are prone to a significant amount of collapse. As can be seen from Figure 3-25, this corresponds to a wet density of $1.9t/m^3$ for the silty sand and $2.1t/m^3$ for the silty clay, based on a lower quartile value. The region where the material is susceptible to collapse is also highlighted on Figure 3-25.

Wet Density		Dry Do	ensity Ratio	
(t/m^3)	Count	Average	Standard	Lower
			Deviation	Quartile
1.5 to 1.6	7	89.2%	3.3%	87.0%
1.6 to 1.7	15	87.7%	2.5%	86.5%
1.7 to 1.8	12	91.0%	4.2%	88.0%
1.8 to 1.9	19	92.0%	4.0%	89.5%
1.9 to 2.0	31	95.1%	3.6%	92.5%
2.0 to 2.1	46	97.3%	3.4%	95.1%
2.1 to 2.2	22	101.0%	1.7%	100.0%
2.2 to 2.3	6	103.9%	2.1%	102.9%
2.3 to 2.4	1	112.5%	-	112.5%
2.4 to 2.5	0	-	-	-

Table 3-5. S	Summary of	Density	Testing –	Silty Sand
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There is a large amount of scatter in the data, as shown on Figure 3-25 and in the standard deviation shown in Tables 3-4 to 3-6. By adopting a lower quartile approach to the assessment; the dry density ratio estimated for a given wet density, will be a somewhat conservative estimate of the dry density ratio. The estimated collapse settlements (or strain) would therefore be an upper limit.

For example, a Silty Clay having a wet density of $2.1t/m^3$ could have a dry density ratio could range from 94% to 101% of the SMDD. Based on a lower quartile approach, the dry density ratio is 95% of the SMDD, which is a somewhat conservative approach.

Wet Density		Dry De	ensity Ratio	
(t/m ³)	Count	Average	Standard	Lower
			Deviation	Quartile
1.5 to 1.6	0	-	-	-
1.6 to 1.7	0	-	-	-
1.7 to 1.8	2	90.0%	9.9%	86.5%
1.8 to 1.9	12	90.3%	4.0%	87.5%
1.9 to 2.0	22	92.6%	2.8%	91.0%
2.0 to 2.1	48	96.7%	2.9%	94.9%
2.1 to 2.2	36	99.6%	2.3%	98.0%
2.2 to 2.3	21	103.8%	1.9%	102.5%
2.3 to 2.4	0	-	-	-
2.4 to 2.5	1	102.0%	-	102.0%

	Table 3-6.	Summary o	f Density '	Testing -	Silty C	lay
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Tube Samples

The calculation of the density from the tube samples has been used to correlate the other density testing techniques. As such, the results of the density calculated from tube samples are discussed further in Section 3.4.

Downhole Gamma Density Testing

The DHGD test results are presented in Appendix 3B, as well as the results of the wet density recorded during the *in situ* density tests carried out within the test pits and from the tube samples.

The results of the DHGD tests generally show that in situ densities are greater than $2.0t/m^3$; however there are some zones of relatively low density.



Figure 3-25. Summary of Density Test Results

Some of these low density zones had a corresponding increase in the caliper width, which indicates that a cavity of some description, which would have possibly been caused by the drilling process, had been intercepted. This means that the low density zone is an artefact of the drilling process rather than a soft zone within the soil.

The results of the DHGD testing were the subject of further investigations which are discussed further in Section 3.4.

Large Scale Test Pits

The purpose of the large scale test pits was to allow direct observation of the subsurface profile to be made and logged and to allow *in situ* density testing to be carried out.

In Areas 4, 7 and 9 the test pits were excavated to assess the likely landform construction methodologies, as well as enabling density testing to be carried out.

As noted above, scrapers are unable to place layers greater than about 300mm in thickness and due to the placement methodology, there would be evidence of discrete parallel layers. Typical layering characteristics generated by truck dumping and spreading are generally, but not necessarily, thicker (greater than 500mm) than layers placed by scrapers, and undulose, i.e. of varying thickness laterally.

Photographs taken during the excavation of the test pits in Areas 4, 7 and 9 are presented on Figures 3-26 and 3-27. Annotations are included on the photographs highlighting observed features used to infer the possible construction methodology.

Noted within test pits located in Area 4 and Area 9 was either an organic odour or a layer of grass matter, which apparently was an old topsoil layer. The presence of the organic material observed within the test pits may have indicated a hiatus in the fill placement in these areas. It was also observed that layering characteristics on either side of the organic material indicated a possible change in the placement methodology. These features are highlighted on Figure 3-27.

Based on observations made of the test pits, the inferred construction methodology for the different Areas are summarised in Table 3-5.



Figure 3-26. Photographs Showing Layering within Test Pit



Figure 3-27. Photographs Showing Layering within Test Pit

The construction methodology has the potential to significantly influence the performance of the landform and its response to dynamic compaction. For example, it is anticipated that that the crater depths and global settlement resulting from dynamic compaction would be larger in areas constructed using truck dumped and spread methodologies than areas constructed using scrapers.

Area	Depth (m)	Inferred Construction Methodology
	0 – 2.2m	Truck placed with spreading
4	2.2m – 2.5m	Previous topsoil layer
	2.5m - >6m	Scraper construction
	0 - 2.9m	Truck placed with spreading
7	2.9m - 4.4m	Scraper construction
	4.4m - >5.9m	Truck placed with spreading
9	0 - >6.3m	Truck placed with spreading

Aethodology

Summary of the Results of the Geotechnical Investigations

Summary plots are presented on Figures 3B-1 to 3B-6 of Appendix 3B detailing the results of the geotechnical testing carried out at each investigation station. Presented on the summary plots is the following information:

- Graph 1 The SPT N value, in 300mm intervals;
- Graph 2 The cone tip resistance (q_c) from the CPT;
- Graph 3 The wet density from the down-hole gamma density test and the *in situ* density testing carried out in the test pits (both wet density and density ratio);
- Graph 4 The caliper width measured by the down-hole gamma density tool; and
- Graph 5 The Dilatometer modulus calculated from each test.

Based on the results presented in Appendix 3B, the following comments about the site characteristics can be inferred:

- The testing results generally indicated that the upper 5m to 7m was of a relatively higher strength material overlying up to 8m of stiff clayey material that, in turn, was overlying shale. Gravels and cobbles were interspersed throughout the profile. The generalised subsurface profile is illustrated in Figure 3-28 below.
- Investigation stations in Areas 8 and 9 generally indicated relatively high SPT N values, comparable to other investigation stations at a similar depth.
- Investigation stations in Areas 4 and 9 typically had a relatively high dry density ratio for a given depth, whereas Area 7 typically had a relatively low dry density ratio. Overall the wet density of the material varied between about 1.8t/m³ and 2.1t/m³.
- There was insufficient data to make generalised conclusions in regards to the flat plate dilatometer test results.



Figure 3-28. Generalised Subsurface Profile

3.4 **Review of Density Testing Procedure**

3.4.1 General

Based on the results of the laboratory testing undertaken (refer to Section 3.2), the importance of accurate assessment of the in situ density was highlighted. Once the in situ density is accurately measured, this can then be related to a dry density ratio from the results of the in situ density testing presented in Figure 3-25 and in Table 3-4. The dry density ratio can then be correlated to a collapse strain using the relationships presented in Figures 3-9 and 3-10.

The assessment of the collapse strain is highly dependent on the accurate measurement of the in situ density. Therefore, further calibration of the DHGD tool is warranted.

The downhole gamma density tool, described in Section 3.3.2, represents a novel approach to the assessment of *in situ* wet density at depth. Due to the sensitivity of the geotechnical model to the in situ density it was necessary to trial the technique to assess its accuracy and hence usefulness to the assessment process. Whilst there was a known sensitivity with the tool which can require repeated testing in order to arrive at reliable values, the DHGD test provided data that could not otherwise be readily obtained. An issue was identified where the variation in borehole diameter, for example due to the presence of gravels or voids in the borehole wall, could lead to misleading low density values.

To calibrate the DHGD tool, further investigation was carried out in the following areas:

- Repeatability of the DHGD testing
- Accuracy of the DHGD testing; and,
- The effect of the rate at which the DHGD probe is removed on the results.

These issues are discussed in the following sections.

3.4.2 Repeatability of DHGD Results

The assessment of the repeatability of the DHGD test results was carried out within boreholes drilled within Areas 4 to 9.

The assessment involved carrying out DHGD testing multiple times within the same borehole, with an attempt to orientate the DHGD tool in different directions. The intention of the multiple tests within a single borehole was to identify isolated, abnormally high density values due to the presence of gravels or other high density materials, or low density values due to localised voids.

Select examples from the DHGD testing are presented in Figures 3-29 and 3-30, with reference to the test results presented in Appendix 3B.

Figure 3-29 provides an example of DHGD testing in which a low density zone was identified during one of the tests. Also indicated on Figure 3-29 is a corresponding change in the borehole width, as denoted by the caliper width. Therefore, it was assessed that the results of Test 1 (on Figure 3-29) between 5m and 8m were influenced by a sidewall intrusion (such as cobbles or gravels) rather than being a low density zone.

Similarly, in Figure 3-30 there is a large decrease in the caliper width corresponding to a significant decrease in the density between a depth range of 5m and 8m. This is assessed as being a sidewall extrusion has affected the density results.


Figure 3-29. Downhole Gamma Density Test Results, Area 8



Figure 3-30. Downhole Gamma Density Test Results, Area 7

In addition to carrying out multiple DHGD tests in a single borehole, an onsite datum borehole (cased) was established. Three DHGD tests were carried out within the datum borehole, immediately after installation, the average of which is used to form a baseline for the datum borehole.

Every time that DHGD testing is carried out on site, the tool is checked for consistency to the previous readings in the datum borehole.

Presented in Figure 3-31 is a comparison of the results of the DHGD testing carried out in the datum borehole on 23 and 31 November 2006; 15 December 2006; and 11 January 2007. Presented in Figure 3-31(a) is a plot of the actual density recorded; whereas presented on Figure 3-31(b) is the difference between the density recorded and the density recorded for the baseline survey. There is a difference in the starting levels for the different datum readings, which is a function of when the testing was started. The depth profile for each has been normalised to a consistent profile.



Figure 3-31. Downhole Gamma Density Test Results, Datum Borehole

The difference between the density recorded and the baseline survey for the different survey records is presented in Table 3-6. The average difference in the testing was

between $0.02t/m^3$ and $0.01t/m^3$. Considering that the density was typically $2.1t/m^3$, then this equates to an average difference of <1%. It is acknowledged that there were some localised differences between readings of up to $0.4t/m^3$ (in absolute terms); however the overall readings indicated that the DHGD testing was providing repeatable results to within 1% of the initial baseline readings.

	Overall	23 Nov. 2006	31 Nov. 2006	15 Dec. 2006	11 Jan. 2007
Average	-0.01 t/m^3	-0.01 t/m^3	-0.02 t/m^3	-0.01 t/m^3	0.02 t/m^3
Diff	0.01 0/11	0.01 0/11	0.02 0.111	0.01 0.111	0.02 0111
Difference					
Standard	0.09	0.09	0.10	0.07	0.07
Deviation					
Maximum	0.38 t/m^3	0.30 t/m^3	0.34 t/m^3	0.38 t/m^3	0.38 t/m^3
Difference					

Table 3-6.	Summary	of Comp	arison b	etween E	Density T	esting
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3.4.3 Accuracy of DHGD Results

The assessment of the accuracy of the DHGD testing results involved carrying out DHGD testing in four boreholes in conjunction with in situ tube sampling (refer to Section 3.3.2 for information regarding the in situ sampling methodology). In addition, large-scale test pits were excavated to facilitate density testing of the soil using nuclear density testing techniques.

The testing methodology noted in Section 3.3.2 was carried out in four boreholes drilled to a depth of 6m within Area 3. At the completion of the drilling, DHGD testing was then carried out in the four boreholes. Two soundings were made within each borehole with the DHGD tool.

At the completion of the DHGD testing and in situ tube sampling within the four boreholes, a large test pit was excavated to enable density testing to be carried out in accordance with Australian Standard AS 1289.5.8.1 (estimation of wet density by nuclear gauge) to depth of up to 6m.

Comparison of the results of the wet density determined by the DHGD testing to the in situ tube sampling and the nuclear gauge is presented in Figures 3-32 and 3-33. A summary of the differences between the testing is presented in Table 3-7.



Figure 3-32. Comparison between DHGD and Nuclear Density Gauge



Figure 3-33. Comparison between DHGD and Tube Sample

	Nuclear Density	Nuclear Density	Tube Samples –
	Gauge – Tube	Gauge – DHGD	DHGD Testing
	Samples	Testing	
Number of Tests	10	52	81
Average	0.11 t/m ³	0.14 t/m^3	0.12 t/m^3
Difference			
Standard	0.1	0.1	0.11
Deviation			
Maximum	0.26 t/m^3	0.34 t/m^3	0.52 t/m^3
Difference			

Table 3-7. Comparison between	Density Tes	sting
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Comparison of the results of the different methods of assessing the in situ wet density indicated the following:

- The density estimated by the Downhole Gamma Density Testing and from the tube samples provided good correlation with the smallest difference in the wet density estimated and the smallest maximum difference. It provides a consistently lower value of the estimate of wet density, which would then provide a slightly conservative estimate of the Dry Density Ratio (when used in conjunction with Figure 3-25).
- The average difference between the different density testing methodologies indicates that a reasonable correlation exists between density testing methods. If it is assumed that the density is typically 2.1 t/m³, then the average difference between density testing is in the order 5%.
- The deviation between the wet density assessed using the nuclear gauge and the other testing methodologies was greater in the upper 3m. Below a depth of 3m, the variation between test results reduces significantly.
- The density of the DHGD testing is sensitive to the diameter of the borehole and possible reaming of the borehole during the drilling process. As noted above, this can indicate possible zones of low density, which were an artifice of the drilling

process rather than an area of low density.

• The nuclear density testing appears to be affected by very dense layers overlying loose zones. This can be seen in Figure 3B-1, for example, where the *in situ* density and Density Ratio (from the nuclear density gauge) indicates a relatively high density at depths of approximately 1.6m to 1.9m, whereas the DHGD tool indicates that an initial density close to that of the nuclear density gauge, however with a significant decrease in density immediately below this (which also corresponds to the results of the tube sample).

A synopsis of the above is that the density testing results appear to be reasonably consistent – within 5% of the test results. The test methods appear to be affected differently by the in situ conditions, and as such it is suggested that at least two of the above methods be undertaken to confirm the magnitude of the test results. The test results indicated that the test methods that have the best correlation appear to be the DHGD and the tube samples.

3.4.4 Rate of DHGD Test

To assess the effect of the rate at which the tool is lifted from the borehole, DHGD testing was carried out within the datum borehole at variable speeds.

Tests were carried out at a rate of approximately 7m/minute and 11 m/minute, the results of which are presented in Figure 3-34. The results in Figure 3-34 were reasonably consistent; indicating that the rate at which the DHGD test was carried out had limited effect on the results obtained.



Figure 3-34. Comparison of Speed of DHGD Test

3.4.5 Discussion

The repeatability of the results was addressed by carrying out DHGD testing in a datum borehole and then carrying out multiple tests within each borehole. A comparison was made between the readings to the baseline data, with the results showing that, on average, the test results were within 1% of the baseline data.

The accuracy of the density testing results was assessed by comparing the results of the DHGD testing to the *in situ* density measured by the nuclear gauge and from that calculated from taking tube samples. Strong correlations exist between the DHGD testing and the results of the in situ tube sampling. The results of the DHGD testing and those of the in situ tube sampling are generally within 5% of the results of the nuclear density testing (on average); and appear to accurately assess the wet density.

The use of DHGD testing was considered an appropriate means of assessing the *in situ* wet density of the subsurface profile before and after dynamic compaction when coupled with *in situ* tube sampling.

3.5 Summary and Conclusions

Based on the results of the laboratory testing, initial classifications tests determined the fill could be classified as a Silty Sand (SM), ranging to low plasticity silt or clay (CL). Only a few samples could be classified as a medium plasticity silt.

Laboratory collapse strain testing was undertaken to identify relationships between the density of the material and the collapse potential (or collapse strain). From the testing it was identified that:

- The collapse strain is a function of the overburden pressure and the dry density ratio at which the material is placed. The results have been presented graphically to show this relationship.
- The soil structure remains stable until inundation, after which settlement occurs rapidly.
- The material type, in particular the fines content, has a significant influence on the magnitude of collapse settlement, with sandy materials indicating less collapse settlement than clayey material.

Investigation into the mechanisms of collapse settlement of the silty sand material using an electron microscope indicated that there was no cementation between particles and that the mechanism involved in collapse was a softening of particle-to-aggregate contacts. The particle-to-particle contact observed before inundation appears to remain intact during the inundation process. There is a small increase in the contact area as result of inundation; however this is assessed as being colloidal material. The change in collapse appears to be dominated by the softening of particle-to-aggregate contacts with a minor contribution to the collapse settlement due to the loss of soil suction between the particle-to-particle bonds.

As for the silty sand material, observation of the silty clay material under the electron microscope, both before inundation and after inundation, indicated a lack of cementation. The particle-to-aggregate and aggregate-to-aggregate contact, in the silty clay material, before inundation also appears to be discrete, however after inundation there was no indication of these contacts existing in discrete form. It is therefore assessed that the binding of the aggregate contacts appear to have softened and collapse of the structure occurred. It is likely that some colloidal materials also fill the void space during the inundation process, as was seen around the more stable particle contacts.

Using this understanding of the behaviour of the fill material, geotechnical investigations were undertaken across the areas in which dynamic compaction is to be carried out. The key outcome of the geotechnical investigations is the fill material varied significantly in terms of its engineering properties and density. However, considering the relatively consistent particle size distribution and Atterberg Limits test results, relationships were established between dry density ratio and wet density of the fill.

From this statistical correlation, the wet density of the fill (measured in situ from the DHGD test) can be correlated to the collapse strain of the fill. However, the repeatability and accuracy of the DHGD testing needed to be confirmed.

The repeatability of the results was addressed by carrying out DHGD testing in a datum borehole and then carrying out multiple tests within each borehole. A comparison was made between the readings to the baseline data, with the results showing that on average, the test results were within 1% of the baseline data.

The accuracy of the density testing results was assessed by comparing the results of the DHGD testing to the in situ density from the nuclear gauge and from the density calculated from taking tube samples. Strong correlations exist between the DHGD testing and the results of the in situ tube sampling. The results of the DHGD testing and the results of the in situ tube sampling are generally within 5% of the results of the nuclear density testing (on average).

The use of DHGD testing was considered an appropriate means of assessing the in situ wet density of the subsurface profile.

Upon confirmation and calibration of the preferred test method, Chapter 4 details how these geotechnical investigation tools are used to quantify the effect of collapsing soils on the landform. Subsequently, in Chapters 5 and 6, details of the dynamic compaction process are given, as well as further review of the effect dynamic compaction has on the ground and the resultant effect on the surface settlement profile.

APPENDIX 3A

RESULTS OF CLASSIFICATION LABORATORY TESTING

TABLE 3B: Summary of Laboratory Testing

Sample	Particle Size	Distribution	Atterberg Limit Test Results		Compac	Natural			
	Те	st					Res	sults	Moisture
	% Passir	ng Sieve							Content
	75□m Sieve	2.36mm	LL	PL	PI	LS	SMDD	SOMC	(%)
		Sieve	(%)	(%)	(%)	(%)	(t/m ³)	(%)	
1	66.7	100.0	27.0	13.0	14.0	6.5	1.92	12.9	13.4
2	41.3	100.0	NP	NP	NP	NP	1.91	12.2	10.7
3	79.4	100.0	33.0	17.0	16.0	4.5	1.70	17.5	19.4
4	72.7	100.0	25.0	13.0	12.0	5.5	1.86	12.5	7.3
5	32.8	100.0	NP	NP	NP	NP	1.76	10.0	8.3
6	57.6	100.0	23.0	12.0	11.0	4.5	1.86	11.2	6.3
7	46.0	100.0	NP	NP	NP	NP	1.90	13.0	11.0
8	20.8	100.0	NP	NP	NP	NP	1.76	13.9	3.7
9	66.7	100.0	28.0	14.0	14.0	8.5	1.88	11.3	6.7
10	52.7	100.0	23.0	15.0	8.0	5.0	1.91	12.4	8.8

Sample	Particle Size	Distribution		Atterberg Limit Test Results		lts	Compac	Natural	
	Te	st					Res	sults	Moisture
	% Passir	ng Sieve							Content
	75□m Sieve	2.36mm	LL	PL	PI	LS	SMDD	SOMC	(%)
		Sieve	(%)	(%)	(%)	(%)	(t/m ³)	(%)	
11	32.0	100.0	NP	NP	NP	NP	1.89	12.5	9.0
12	68.9	100.0	25.0	15.0	10.0	1.5	1.87	12.8	10.2
13	75.0	100.0	34.0	14.0	20.0	10.5	1.78	13.9	9.3
14	55.0	100.0	26.0	14.0	12.0	3.5	1.89	12.2	5.6
15	50.4	100.0	26.0	13.0	13.0	5.5	1.87	12.1	6.7
16	54.3	100.0	NP	NP	NP	NP	1.88	11.9	6.6
17	64.8	100.0	20.0	16.0	4.0	0.5	1.87	12.6	9.8
18	71.4	100.0	23.0	15.0	8.0	3.5	1.87	14.5	12.8
19	49.1	100.0	NP	NP	NP	NP	1.81	10.7	5.4
20	59.3	100.0	22.0	14.0	8.0	4.5	1.93	11.5	8.1
21	85.0	100.0	37.0	17.0	20.0	10.0	1.72	17.3	13.0

Sample	Particle Size	Distribution		Atterberg Limit Test Results			Compac	Natural	
	Te	est						Results	
	% Passi	ng Sieve							Content
	75□m Sieve	2.36mm	LL	PL	PI	LS	SMDD	SOMC	(%)
		Sieve	(%)	(%)	(%)	(%)	(t/m ³)	(%)	
22	44.0	100.0	NP	NP	NP	NP	1.87	11.2	4.9
23	55.3	100.0	NP	NP	NP	NP	1.88	10.8	6.1
24	64.9	100.0	24.0	14.0	10.0	3.5	1.73	12.6	9.7
25	60.4	100.0	NP	NP	NP	NP	1.87	13.3	8.3
26	44.6	100.0	NP	NP	NP	NP	1.81	13.4	5.0
27	30.2	100.0	NP	NP	NP	NP	1.79	10.3	6.3
28	60.6	100.0	NP	NP	NP	NP	1.78	14.8	11.0
29	58.1	100.0	NP	NP	NP	NP	1.85	11.9	7.0
30	61.0	100.0	21.0	15.0	6.0	1.0	1.89	12.4	6.4
31	48.9	100.0	NP	NP	NP	NP	1.89	11.5	6.0
32	49.2	100.0	NP	NP	NP	NP	2.01	12.1	6.4

Sample	Particle Size	Distribution		Atterberg Limit Test Results			Compac	Natural	
	Te	st					Res	sults	Moisture
	% Passi	ng Sieve							Content
	75□m Sieve	2.36mm	LL	PL	PI	LS	SMDD	SOMC	(%)
		Sieve	(%)	(%)	(%)	(%)	(t/m ³)	(%)	
33	72.6	100.0	28.0	17.0	11.0	6.5	1.83	12.3	7.7
34	35.7	100.0	NP	NP	NP	NP	1.84	113.6	4.2
35	30.4	100.0	NP	NP	NP	NP	1.79	12.7	5.5
36	43.4	100.0	NP	NP	NP	NP	1.80	13.4	6.6
37	86.1	100.0	27.0	15.0	12.0	7.5	1.78	14.9	12.3
38	78.4	100.0	26.0	14.0	12.0	3.5	1.81	14.8	10.9
39	90.8	100.0	32.0	18.0	14.0	6.0	1.79	14.8	13.2
40	74.6	100.0	22.0	15.0	7.0	5.0	1.86	13.5	10.9
41	79.7	100.0	30.0	15.0	15.0	7.5	1.79	15.0	12.0
42	63.8	100.0	NP	NP	NP	NP	1.86	14.5	15.2
43	54.1	99.6	NP	NP	NP	NP	1.86	12.6	9.1

Sample	Particle Size	Distribution st	Atterberg Limit Test Results				Compac Res	Natural Moisture	
	% Passir	ng Sieve							Content
	75□m Sieve	2.36mm	LL	PL	PI	LS	SMDD	SOMC	(%)
		Sieve	(%)	(%)	(%)	(%)	(t/m ³)	(%)	
44	26.0	94.2	NP	NP	NP	NP	1.93	12.0	10.7
45	33.3	100.0	NP	NP	NP	NP	1.84	13.7	6.0
46	51.8	100.0	21.0	15.0	6.0	0.0	1.95	10.8	7.0
47	69.9	100.0	27.0	13.0	14.0	4.5	1.83	13.6	7.3
48	44.1	100.0	NP	NP	NP	NP	1.79	14.4	5.3
49	14.8	59.9	NP	NP	NP	NP	2.24	6.7	2.6
50	33.5	94.9	NP	NP	NP	NP	1.96	11.3	10.5

Note: NP-Non-Plastic

LL – Liquid Limit

PL – Plastic Limit

PI – Plasticity Index

LS – Linear Shrinkage

SMDD – Standard Maximum Dry Density

SOMC – Standard Optimum Moisture Content

APPENDIX 3B

RESULTS OF GEOTECHNICAL INVESTIGATIONS



Figure 3B-1. Summary of Results – Area 4



Figure 3B-2. Summary of Results – Area 5



Figure 3B-3. Summary of Results – Area 6



Figure 3B-4. Summary of Results – Area 7



Figure 3B-5. Summary of Results – Area 8



Figure 3B-6. Summary of Results – Area 9

CHAPTER FOUR

EFFECT OF COLLAPSE SETTLEMENT

4 EFFECT OF COLLAPSE SETTLEMENT

4.1 Introduction

As detailed in Chapter 3, laboratory testing was undertaken to develop an understanding of the mechanisms causing collapse settlement in the dominant materials found on the site. Further laboratory work was undertaken to quantify the magnitude of collapse strains as a function of dry density ratio (of the Standard Maximum Dry Density) and material type.

A database of density testing was then used to establish statistically meaningful relationships between dry density ratio and wet density. By adopting a lower quartile trend line for the Silty Sand and Silty Clay, a linear relationship between dry density ratio and wet density was developed. Therefore, relationships could be developed between collapse strains and the wet density of the material.

The final part of Chapter 3 focused on calibrating techniques to measure the in situ density; i.e. using Downhole Gamma Density and taking Tube Samples, to establish a density profile with depth. These two tests formed a significant part of the testing regime and establishing the link between the in situ profile and the collapse potential.

While a lot of the focus has been on establishing test methods to assess the density profile, Flat Plate Dilatometer testing and CPT probes have also been incorporated into the testing regime to better characterise the soil profile (in terms of material type) and also quantify the stiffness of the soil profile with depth.

Using these test methods, geotechnical investigations were carried out across 6 different Areas (refer to Chapter 3, Section 3.3.2); the results of the investigation are presented on summary plots in Appendix 3B of Chapter 3. The generalised subsurface profile was shown on Figure 3-28, and is repeated below as Figure 4-1.

The results of the geotechnical investigations, presented in Chapter 3, identified randomly distributed zones of low density material both laterally across the site and at different depths. For example, the results presented on Figure 3B-2 (in Appendix 3B, of Chapter 3) show that there are distinctive zones where the density is less than 1.9t/m³, hence are susceptible to collapse. The zones of low density do not appear to be continuous and were not observed in any of the other investigation locations.



Figure 4-1. Generalised Subsurface Profile

Observation of the side wall profile of large scale test pits excavated as part of the investigation (refer to Section 3.3.3 of Chapter 3, specifically Figures 3-26 and 3-27) shows the undulated layering profile. Based on the observation of the test pits, these layers tend to extend for a lateral distance of approximately 10m – however some of the layering extended beyond 15m (the width of the test pit).

The effect that the zones that are prone to collapse have on the ground surface profile is the subject of this chapter. In Section 4.2 the methodology used to assess the effect of the collapse zones is discussed in detail, with Section 4.3 focusing on the calculation of landform performance based on the results of the geotechnical investigation presented in Appendix 3B (of Chapter 3).

4.2 Method of Assessment

4.2.1 General

The effect of zones of collapse settlement on the ground surface has been assessed based on research undertaken by Burland *et al.* (2002) for the construction of the Jubilee Line (tube tunnel) in London to estimate the ground surface profile that would occur as a result of the tunnelling work. Burland *et al.* (2002) also considered the effect of the building stiffness and correlated ground movements to a building strain and "damage category".

The effect of each of the terms in the Burland *et al.* (2002) model on the ground surface (settlement) profile has been the subject of a numerical study. Furthermore, additional

work has been undertaken to extrapolate he model presented by Burland *et al.* (2002) to collapsing zones within a filled site profile. The results of the numerical study are presented in the following sections.

4.2.2 Ground Settlement Profile

The ground surface profile expected as a result of collapse settlements is assessed using work carried out by Burland *et al.* (2002) regarding the risk of building damage due to tunnelling. Burland *et al.* (2002) presented a model which is based on a volumetric strain due to tunnelling effects such as movement of the ground onto the face of the tunnel and radial movement towards the tunnel axis due to reductions in lateral earth pressures.

Presented in Figure 4-2 is an illustration of the Burland et al. (2002) model.



Figure 4-2. Simplified Design Model

The slope of the surface profile can be calculated as either the "hogging" slope, where the footing will be spanning over the settlement profile, or the "sagging" slope, where the footing is bending with the settlement profile, as shown on Figure 4-3.

For the purpose of assessing the ground surface profile resulting from collapse settlements, the distinction between a hogging or sagging slope is not required. More so, the analyses are concerned with the largest of the two slopes and hence the largest deflection ratio, Δ/L .



Figure 4-3. Schematic Diagram of Slope of Settlement Profile

The settlement profile is based on an empirical approach to predict surface settlements as described by Burland (1999) and Burland *et al.* (2002). The settlement profile is based on a Gaussian distribution and is given by the following equations:

$$S_{MAX} = 0.31 V_L \frac{D^2}{KZ_0}$$
[4-1]

$$S_{y} = S_{MAX} e^{\frac{-y^{2}}{2i^{2}}}$$
[4-2]

Where: S_{MAX} = maximum settlement;

 S_y = surface settlement at distance y from centre of the tunnel;

y = horizontal distance from centre of the tunnel;

V_L =volumetric strain;

D = diameter of the tunnel;

i = point of inflection = K.Zo;

Zo = depth to centre of tunnel; and,

K = trough parameter.

The model presented by Burland (1999) and Burland *et al.* (2002) is being extended to represent a partially collapsible zone with the following parameters:

- The collapse strain of the collapsible zone is equivalent to Burland's volumetric strain, V_L;
- The height of the collapsible zone is equivalent to Burland's tunnel diameter, D; and
- The depth to the midpoint of the collapsible zone is equivalent to Burland's depth to the centre of the tunnel, Zo.

The ground surface profile calculated using the "extended" Burland model is particularly sensitive to certain assumptions and input parameters. These are:

- The collapse strain;
- The trough parameter;
- The width of the collapsible zone; and,
- Selection of equivalent diameter of the zone undergoing collapse.

Each of the above is discussed in more detail in the following sub-sections.

Collapse Strain

For this study, the collapse strain, derived from the laboratory testing detailed in Chapter 3, is assumed to be equivalent to Burland's volumetric strain.

The volumetric strain, V_L , is a parameter used by Burland *et al.* (2002) to address a variety of effects resulting from tunnelling. These include "*movement of ground into the face of the tunnel and radial movement towards the tunnel axis due to reductions in supporting pressures*". They note that the volumetric strain is dependent on the type of ground, the groundwater conditions, the tunnelling method, the length of time in providing positive support, and quality control measures.

The collapse strain is material and stress dependent similar to the volumetric strain described by Burland *et al.* (2002). In both the tunnelling and collapse settlement cases, there is a contraction of a void, such as a tunnel, that causes settlement in the overlying soil. The propagation of the contraction to the surface, and the shape of the settlement profile at the surface, is governed by the trough parameter.

Trough Parameter

The trough parameter, K, is typically 0.3 for sand and 0.5 for clay materials (Burland *et al.*, 2002). Clayey materials tend to arch or bridge over the voids, hence have a much higher trough parameter. Sands, which tend to move into a void more readily than clays, have a much lower trough parameter and result in settlement profile that has a much higher deflection ratio, Δ/L (refer to Figure 4-3).

Based on the laboratory testing carried out at the site, which shows a material range typically between a silty sand and a silty clay, a K value of 0.4 has been assumed in the analyses presented below.

Width of the Collapsible Zone

The Burland model assumes that the zone undergoing a volumetric strain is of equal width and depth (i.e. circular), which is generally applicable for tunnels. However, this may not necessarily be the case for filled sites where the material would typically be placed in layers.

As discussed in Section 4.1; and presented in Figures 3-26 and 3-27 in Chapter 3; the construction of the landform has resulted in a layered soil profile. The thickness and width of the layers depends on the construction methodology; however they have been noted to be 10m wide (or larger in some cases) and nominally 2m thick.

An investigation was undertaken to assess the effect the width of the collapsible zone has on the ground surface profile by assuming a series of collapsible zones at the same depth, with the resultant ground settlement profile for each individual zone superimposed to calculate the cumulative effect of the collapse. The assessment method is shown schematically in Figure 4-4.

Sensitivity studies were carried out looking at the effect the following parameters have on the resultant ground surface profile:

- The depth to the midpoint of the collapse zone (Zo);
- The diameter of the collapse zone, D; and
- The volumetric strain.





For the sensitivity studies, the material was assumed to be between a clay and sand, and hence a K of 0.4 was adopted. The study considered elastic settlements so that the ground settlement profile of each individual circular collapse zone could be simply superimposed and the cumulative settlement profile calculated.

As a result of the analyses, a series of plots were presented that show the relationship between X (the width of the collapse zone); the diameter of the collapse zone, and the maximum deflection ratio for a given depth. The maximum deflection ratio has been normalised with respect to the maximum deflection ratio occurring for a single circular collapsible zone, i.e. when X = 1. By way of example, the maximum deflection ratio occurring for a collapsible zone that is 1m high and 5m wide (X = 5) is 12.7 times the maximum deflection ratio if X = 1.

An example of the outcome of the assessment is presented in Figure 4-5, below, for the depth to the midpoint of the collapsible zone being 5m. For the different diameters of collapsible zone, the slope multiplier appears to reach a maximum value; and as the width of the collapse zone increases (i.e. X increases) the maximum deflection ratio decreases. As can be seen on Figure 4-5, for a 1m diameter collapsible zone, the slope multiplier increases up to X = 8; after which the multiplier decreases.



Figure 4-5. Slope Multiplier, for a Depth of 5m

The trend where the slope multiplier reaches a maximum value was observed for each of the different depths analysed, not just at a depth of 5m as shown in Figure 4-5. By normalising the slope multiplier for a range of diameters of collapsible zones, and for different depths, a design profile can be developed, as shown on Figure 4-6.





The design profile considered in Figure 4-6 can be expressed as follows:

$$\frac{Slope\ Multiplier}{Z_0} = 3.3D^{-1.5}$$
[4-3]

Therefore, by knowing the depth and the diameter of the collapsible zone, Equation [4-3] can be used to calculate the slope multiplier. The slope multiplier is used to calculate the maximum ground surface slope that would coincide with the maximum width of a collapsible zone.

Diameter of Zone Undergoing Collapse

A study has been carried out to assess the effect of the diameter of the zone undergoing collapse on the ground surface slope (or differential settlement at the ground surface level).

For the study, a 12m deep fill profile with the potential for collapse was examined. The surface settlement profile, and corresponding ground surface slope, was assessed by varying the number of layers, N, in the zone of fill with potential for collapse. A summary table presenting the number of zones and diameter of zones is presented in Table 4-1 and illustrated in Figure 4-7.

Number of Layers, N	Diameter of Each Layer, D	Total thickness
1	12m	12m
2	6m	12m
3	4m	12m
4	3m	12m
6	2m	12m
12	1m	12m

Table 4-1	Diameter	of Zones	undergoing	Collanse
1 auto 7-1.	Diameter	UI LUIIUS	unucigoing	Conapse

The results of the study are presented in Figure 4-8 in the form of a plot showing the maximum deflection ratio divided by the percentage of collapse strain versus the diameter of the zone undergoing collapse.



Figure 4-7. Model for Diameter of Collapsible Zone

Therefore, the fill history and the layering of the fill placement (refer to Chapter 3) become an import consideration. Where the fill was placed in thick layers using truck dumping and spreading, there is the potential for large diameter collapsible zones to form. Conversely, when scrapers are used, the layer thickness is limited to about 300mm, thus not allowing large diameter zones of collapse to form.



Figure 4-8. Plot of Relationship between Slope, Collapse Strain and Diameter

4.2.3 Effect on Buildings

The impact of differential settlements on buildings was investigated by Burland *et al.* (2002) as part of their work into ground loss associated with the construction of a tunnel. They presented relationships that equate the deflection ratio, Δ/L , at which cracking is initiated, which is discussed below.

Timoshenko (1957) provided the general form of deflection, Δ , at the mid-span of a centrally loaded beam having both bending and shear stiffness as follows:

$$\Delta = \frac{PL^3}{48EI} \left[1 + \frac{18EI}{L^2 HG} \right]$$
[4-4]

Where E = Young's Modulus (MPa)

G = Shear Modulus (MPa)

- L = Length of Beam (m)
- H = Thickness of Beam (m)
- I = Second Moment of Area (m²)
- P = Point Load (kN)

Burland *et al.* (2002) presented Equation 4-4 in terms of the deflection ratio and maximum extreme fibre strain, ε_{bMAX} , as follows:

$$\frac{\Delta}{L} = \left[\frac{L}{12t} + \frac{3I}{2yLH}\frac{E}{G}\right]\varepsilon_{bMAX}$$
[4-5]

- Where t = vertical distance to the edge of the neutral axis, assumed to be half the building height (assumed to be 1.5m for a single-storey, 3m for a two storey house).
 - H = Height of the Building (3m for a single storey house, 6m for a two-storey house)
 - L = Length of Building (m)
 - y = horizontal distance from the edge of the slab.

Burland *et al.* (2002) also presented Equation 4-4 in terms of the maximum diagonal strain, ε_{dMAX} , as follows

$$\frac{\Delta}{L} = \left[1 + \frac{HL^2}{18I}\frac{G}{E}\right]\mathcal{E}_{dMAX}$$
[4-6]

Consideration of the influence of the horizontal strain, ε_h , was included by Boscardin and Cording (1989) by superposition. The resultant extreme fibre strain, ε_{br} , and resultant

diagonal strain, ε_{dr} , can be expressed as follows:

$$\varepsilon_{br} = \varepsilon_{bMAX} + \varepsilon_h \tag{4-7}$$

$$\varepsilon_{dr} = \varepsilon_h \left[\frac{1 - \upsilon}{2} \right] + \sqrt{\varepsilon_h^2 \left[\frac{1 + \upsilon}{2} \right]^2 + \varepsilon_{dMAX}^2}$$
[4-8]

Therefore, in order to assess the resultant extreme fibre strain and the resultant diagonal strain on a building the first step is to quantify the likely ground movement profile resulting from a collapse zone, which is described in Section 4.2.2. Based on the ground surface profile, calculate the hogging and sagging deflection ratios. Also using the ground movement profile, the horizontal strain can be calculated. Using the maximum deflection ratio and Equations [4-5] and [4-6], the maximum extreme fibre strain and maximum diagonal strain is calculated. The resultant strain can be calculated using Equations [4-7] and [4-8].

Once the resultant building strains are calculated, this can be compared to recommended values that compare to damage categories; which is discussed in the following section.

4.2.4 Classification of Damage

The Classification of Damage to a structure is outlined in Australian Standards, such as AS 2870 - 1996, and Burland *et al.* (2002) for example. Presented in Table 4-2 is a summary of the Damage Category system of the Australian Standard.

Boscardin and Cording (1987) initially looked at the tolerance of structures to excavationinduced ground movements including the effect of tunnels by comparing the results of case studies to analytical studies. Boscardin and Cording (1987) used earlier work by Burland and Wroth (1974) on the bending and shear of a simply supported beam to deduce that for structures with a ratio of length to height less than 1 (which is typical for single and two storey houses) then the "*first damage will be in the form of diagonal tension cracking*". By including horizontal ground strain, which occurs as a result of excavations and tunnelling, Boscardin and Cording (1987) note that a smaller amount of differential settlement or angular rotation is required to initiate damage to the building. Therefore, they postulated that the critical strain is the sum of the maximum fibre strain and the horizontal strain.

Description		Damage
Reference to Walls	Reference to Concrete Floors	Category
Hairline cracks.	Hairline cracks, insignificant	0
Cracks <0.1mm	movement of slab from level.	
	Cracks in floor <0.3mm;	
	Change in offset from a 3m	
	straight edge <8mm.	
Fine cracks which do not need repair.	Fine but noticeable cracks. Slab	1
Cracks <1mm	reasonably level.	
	Cracks in floor <1mm; Change	
	in offset from a 3m straight edge	
	<10mm.	
Cracks noticeable but easily filled. Doors and	Distinct cracks. Slab noticeably	2
windows stick slightly.	curved or changed in level.	
Cracks <5mm	Cracks in floor <2mm; Change	
	in offset from a 3m straight edge	
	<15mm.	
Cracks can be repaired and possibly a small amount	Wide cracks. Obvious curvature	3
of wall will need to be replaced. Doors and	or change in level.	
windows stick. Service pipes can fracture. Weather	Cracks in floor 2mm to 4mm;	
tightness often impaired.	Change in offset from a 3m	
5mm to 15mm cracks (or a number of cracks 3mm	straight edge 15mm to 25mm.	
or more in one group).		
Extensive repair work involving breaking-out and	Gaps in slab. Disturbing	4
replacing sections of walls, especially over doors	curvature or change in level.	
and windows. Window and door frames distort.	Cracks in floor 4mm to 10mm;	
Walls lean or bulge noticeably, some loss of bearing	Change in offset from a 3m	
in beams. Services pipes disrupted.	straight edge >25mm.	
15mm to 25mm cracks, also dependent upon		
number of cracks		
L	l	I

Table 4-2. Damage Class System (from AS 2870 – 1996)

Notes: 1

2

Crack width is the main factor by which damage to walls is categorised. The width may be supplemented by other factors, including serviceability, in assessing category of damage. In assessing the degree of damage, account shall be taken of the location in the building or structure where it occurs, and also the function of the building or structure.
- 3 Where the cracking occurs in easily repaired plasterboard or similar clad-framed partitions, the crack width limits may be increased by 50% for each damage category.
- 4 Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.
- 5 Account should be taken of the past history of damage in order to assess whether it is stable or likely to increase.
- 6 The straight edge is centred over the defect, usually, and supported at its ends be equal height spacers. The change in offset is then measured relative to this straight edge.

Presented in Figure 4-9 is a graph developed by Boscardin and Cording (1987) relating horizontal extension, angular distortion (or deflection ratio, Δ/L) and degree of damage.



Figure 4-9. Relationship of Damage to Angular Distortion and Horizontal Extension (Boscardin and Cording, 1987)

Burland and Wroth (1974) suggested the range to critical tensile strain for very slight damage to be 0.00075, which corresponds to Damage Category 1 of Table 4-2. Slight damage (or Damage Category 2, in Table 4-2) is taken at a critical tensile strain of 0.0015, which Boscardin and Cording (1987) note corresponds to an angular distortion of 1/300. Moderate to Severe damage (Damage Category 3) is taken as critical strains up to 0.0030.

AS 2870 – 1996 states "for most situations Category 0 or 1 should be the limit, however, under adverse conditions, Category 2 should be expected although such damage is rare". Similar expectations of damage categories are presented in Burland *et al.* (2002) limiting the cracking of the houses to a Damage Category of 1.

4.2.5 Summary of Assessment Process

Based on the analytical study, and considering the effect the width and diameter of the collapsible zone, the following is a summary of the assessment process:

- Develop a geotechnical model based on collapsible zones within the fill (considering the fill construction history) and Volume Loss (Collapse Strain) parameters;
- Calculate the sagging and hogging ratio for each discretised layer and using the Slope Multiplier in Equation [4-3] calculate the maximum slope;
- From Equations [4-4] and [4-5], calculate the maximum extreme fibre strain and maximum diagonal strain;
- From Equations [4-6] and [4-7], calculate the resultant fibre strain and diagonal strain;
- Compare the theoretical fibre and diagonal strains to the limiting strains suggested by Burland; and,
- Calculate the resultant Damage Category for single and two storey buildings.

Using this methodology, a basis has been developed for assessing the effect of collapsible soils on the ground surface profile, and also the building strains induced as a result of the ground movement. It should be noted that these results have not been verified and calibrated with laboratory (or field) tests or numerical modelling. This limitation is highlights in Section 7.2 (Future Research).

Based on the results of the numerical study, a design plot has been developed that relates the size of the collapsible size and depth of the collapsible zone to the maximum settlement and also Damage Categories. This is presented in Figure 4-10 below.

Figure 4-10 is similar to that presented by Boscardin and Cording (1987) however has included the methodology described by Burland *et al.* (2002) and extended to address collapsing soils.



Figure 4-10. Summary Plot of Analysis

4.3 Assessment of Current Landform

Based on the results of the geotechnical assessment presented in Chapter 3, an assessment of the landform performance at different investigation locations across the Areas has been carried out. The results are summarised in Table 4-2.

Assessment Category		Count	Maximum	Average	Standard
					Deviation
Slope Induced by Settlement		9	1 in 1,755	1 in 12,146	1 in 11,668
Strain induced	Single Storey	9	0.103%	0.038%	0.036%
by Settlement	Double Storey	9	0.121%	0.048%	0.040%
Damage Category	Single Storey	9	2	0.56	0.88
	Double Storey	9	2	0.67	1.00

The results of the assessment show that, firstly, there is a large variability in the results calculated. This is consistent with the site comprising of uncontrolled fill. While the

material properties vary marginally; the density of which the soil was placed varies significantly.

It is also worth noting, that the maximum building strains calculated could be in excess of 0.1% (corresponding to an unacceptable level of damage, being Damage Category 2). This, in conjunction with the large variance in results, confirms the need for some form of treatment to compact the soil and reduce the amount of settlement occurring from collapsible soils.

Some areas encountered exhibited a high density. These areas are likely to have been placed and compacted using scrapers (as discussed in Chapter 3). The effect is twofold – not only is the density higher, but the layer thickness is generally much thinner (when compared to truck-placed construction) hence the potential for the formation of a large collapsible zone is very low. This results in a low building strain, as expected.

4.4 Summary and Conclusions

The results of the analyses have shown the effect of the highly variable nature of the fill material, in terms of density and stiffness, and the impact this has on overlying buildings. The building strains (and damage category) range from minimal damage through to strains in excess of 0.1% (Damage Class 2).

It should be noted that with an uncontrolled fill site, there is uncertainty about the representativeness of the results from the site investigation. Without some basis for extrapolation (such as geological processes in a natural site, or an engineered specification in a filled site) there remains a large amount of possible variability in the results; sometimes larger than what is encountered in the geotechnical investigations.

Therefore, there is a need to undertake dynamic compaction to provide a controlled amount of energy being delivered to the site, which will, not only densify the ground, but also improves the uniformity (or homogeneity) of the profile and provide confidence in extrapolating the results of the geotechnical investigations.

Chapter 5 provides detail of the application of dynamic compaction (through number of blows and grid spacing); with Chapter 6 providing further explanation of the effect of dynamic compaction on the soil.

CHAPTER FIVE

DYNAMIC COMPACTION

5 DYNAMIC COMPACTION

5.1 **Dynamic Compaction Works**

5.1.1 General

Dynamic compaction was carried out across the 9 different Areas using the following arrangement:

- The pounder was octagonal shaped, 1.8m wide, and weighed 20 tonnes;
- The drop height was 23m;
- The dynamic compaction was undertaken in three phases;
- The spacing between drop locations was 4.5m; and,
- The pounder was dropped 16 times at each drop location (refer below).

The pounder shape and size was consistent with that presented by Lee and Gu (2004) (Figure 2-16 of Chapter 2) who investigated the optimal size of the pounder. From their results, for a drop energy of 200t.m (marginally less energy than being used in this study) the optimal pounder size lies between 1.6m and 2.0m. Therefore, the adoption of a 1.8m wide pounder is consistent with these findings.

Full scale trials were undertaken in order to refine the dynamic compaction process (or make the application of dynamic compaction more efficient). The main areas of investigation involved assessing:

- The number of blows required at each drop location; and,
- The effect of changing the grid spacing between drop locations.

In the literature, there have not been any studies published to date that investigate these two areas and present the results of full scale trials. Detailed discussion and presentation of the results of the assessment are presented in Section 5.2, for the assessment of the number of blows, and Section 5.3, for the assessment of the grid spacing.

Further studies of the ground response to dynamic compaction have also been undertaken. Presented in Section 5.4 are the results of a detailed investigation of using a topographic survey to monitor the effects of dynamic compaction. At completion of the dynamic compaction process, analyses were carried out to assess the performance of the landform post dynamic compaction using the methodology described in Chapter 4. The results of the analyses are presented in Section 5.5.

Chapter 6 then presents more detail into understanding the subsurface response to dynamic compaction in more detail.

In order to carry out the aforementioned assessment, monitoring of the dynamic compaction works was undertaken. Details of the different monitoring techniques are presented in Section 5.1.2; and a brief description of the geotechnical investigations undertaken subsequent to dynamic compaction is presented in Section 5.1.3.

5.1.2 Monitoring of Dynamic Compaction

5.1.2.1 General

The dynamic compaction process was monitored via recording a series of indicators, which are as follows:

- <u>Record of Penetration</u> A record was made of the number of drop locations carried out, the number of blows at each drop location and the diameter and depth of the resultant crater;
- <u>Heave / Penetration Tests</u> Survey stations were established around a drop location to assess localised heave (and the depth of penetration) resulting from the dynamic compaction process. Further discussion about the heave test is presented in Section 4.2.2.
- <u>Extensometer Monitoring</u> A topographical survey prior to the commencement of the trial and at the completion of the works was carried out.

These test methods are discussed in the following sections.

5.1.2.2 Record of Penetration

The record of penetration is a simplistic method of monitoring the ground response to dynamic compaction. Dynamic compaction delivers a consistent amount of energy to the ground; therefore variations to the depth and diameter of the craters formed during dynamic compaction provide an indication of the ground conditions prior to undertaking dynamic compaction.

A record of penetration simply involves recording the depth and diameter of each crater formed by the dynamic compaction process.

5.1.2.3 Heave / Penetration Testing

Heave / Penetration testing (referred to as "Heave Testing") is carried out at the commencement of each pass of dynamic compaction within a designated treatment area. Heave testing is also undertaken where the record of penetration indicates that there is a substantial variability in the way the ground is responding to dynamic compaction. If a treatment area has a reasonably consistent depth of crater then the number of heave tests required is small. Conversely, when there is a wide range in the depth of each crater for a given number of blows, then the number of heave tests required in a given area is large.

The heave test generally involves establishing 12 survey stations around a drop location along three equally spaced axes. Four stations are also established on four corners of the pounder to monitor the depth of penetration. This configuration is shown on Figure 5-1.



Figure 5-1. Configuration of Heave Test (from Menard)

The survey stations were monitored prior to the commencement of dynamic compaction and then after every 4 blows up to a maximum of 40 blows. In addition, the diameter of the resultant crater, at ground surface level, was generally monitored after every 4 blows.

The results of the heave tests were illustrated via three graphs, namely:

- A plot presenting the volume of heave recorded (calculated from monitoring the 12 survey stations) with blow count;
- A plot presenting the volume of the crater formed by the dynamic compaction works with blow count (calculated from monitoring the survey stations on the pounder and the diameter of the crater); and
- A plot presenting the net volume with blow count; that is the difference between the heave and crater volume (further detail is provided in Section 5.2.2).

Generally, the plot of the net volume initially exhibited a near-linear relationship with blow count, but the change in net volume should diminish with increasing blow count as the ground becomes more dense as a result of the dynamic compaction process. A sketch showing a typical plot of the net volume with blow count is presented in Figure 5-2.



Figure 5-2. Typical Plot of Heave Test Results

The plot of the net volume with blow count is of particular importance as it allows a qualitative assessment to be made regarding the ground improvement being achieved and also provides guidance to determining the required number of blows to be carried out in a given area.

5.1.2.4 Extensometer Monitoring

Extensometer monitoring involves the installation of magnets along a hollow tube, which can be monitored to assess ground movements (both settlement and heave) as a result of the dynamic compaction process. A schematic diagram showing the configuration of the magnets that form the extensometer is presented in Figure 5-3.





A borehole is initially drilled at the midpoint between drop locations on the dynamic compaction grid and the extensometer tube and magnets installed. A probe is then lowered down the inner tube to record depth, from a known datum, of each magnet. Readings are then taken after each pass of dynamic compaction to record the amount of heave and settlement resulting from each pass.



A typical result of the extensioneter monitoring is presented in Figure 5-4. As can be seen, there are distinct zones of heave (or negative settlement) and compression (or settlement).

Figure 5-4. Typical Result of Extensometer Monitoring

Extensometers were installed mid-way between drop locations of the dynamic compaction grid – approximately 2.25m from the centre of each drop location. During the dynamic compaction works, systems were put in place to protect the instrumentation from damage and allow monitoring to be undertaken.

5.1.3 Geotechnical Investigations

Geotechnical investigations were undertaken after dynamic compaction works were completed. The various geotechnical investigation techniques are discussed in detail in Chapter 3; but the methods used to assess dynamic compaction are:

- An Electric Friction Cone Penetrometer Test (CPT);
- An augered Borehole;
- Flat Plate Dilatometer Tests (DMTs) in the borehole;
- In situ Tube Sampling in the borehole; and
- Down Hole Gamma Density (DHGD) Test.

5.2 Assessment of Number of Blows

5.2.1 General

As noted in Section 5.1.2.3, the heave test is a tool used to assess the number of blows required to compact the ground, however in order to refine the dynamic compaction process a detailed review of the number of blows required at each drop location has been undertaken. The review is carried out in two stages, namely:

- 1. A review of the characteristics of the heave tests to identify other means by which the number of blows can be assessed.
- 2. Carry out an assessment of the resultant landform at the reduced number of blows and the existing number of blows to compare the difference between the number of blows adopted.

5.2.2 Background

The methodology of setting out the heave test has been previously described in this dissertation, however to establish the context of this assessment it is discussed further in this section.

The methodology for carrying out heave tests was to monitor three sets of four pegs at ground surface level, and to monitor the penetration of the pounder and diameter of the crater at typically between 2 blow and 4 blow intervals.

The results of the heave test were then plotted on a graph that showed the amount of heave recorded at the ground surface level, the volume of the crater created by carrying out the dynamic compaction and also the net change in volume (i.e. the volume of penetration minus the amount of heave). In a granular, near-homogenous (in terms of material type) subsurface profile, the number of blows to compact the ground is generally determined when the net change in volume reaches a plateau, i.e. when the amount of heave equals the amount of penetration.

The initial heave test results at this site generally indicated that the net change in volume steadily increased with blow count, thus indicating that a large number of blows would be required to compact the ground. Based on observations made, the method of recording and analysing the data from the heave test were not fully indicative of the observed results of

the dynamic compaction process.

To better assess the dynamic compaction process, both the on-site recording methodology and the assessment methodology were revised.

5.2.3 On-site Recording Methodology

Field observations recorded during the course of the initial dynamic compaction work identified two main issues:

- A significant amount of heave occurred between the edge of the crater and the first peg of the heave test resulting in the underestimation of heave; and,
- The diameter of the crater formed during the dynamic process was typically tapered (narrower at the bottom than at the top) and not cylindrical as initially assumed, resulting in an over-estimation of crater volume.

The result of underestimating the amount of heave and overestimating the volume of the craters was to potentially overestimate the number of blows required to compact the ground.

To address the issue, two measures were implemented, namely:

- Refinement of the estimation of the volume of heave being recorded, by using five pegs instead of four as part of the heave test. The first peg was located 1.5m from the centre of the drop location.
- Change of the measuring procedure for recording the dimensions of the crater to record both the upper and lower diameters of the crater.

For the assessment presented herein (and future monitoring) the above revised field trial was implemented.

5.2.4 Assessment of Heave Test Data

The basic premise of the dynamic compaction process is that if there is penetration of the pounder and no heave; then the ground is being compacted. When there is heave (and where the amount of heave equals the amount of penetration) then the ground is being displaced rather than compacted. In order to refine the dynamic compaction process, particularly in highly variable, uncontrolled fill, this basic premise was investigated further.

A review of the assessment of the heave data was carried out with the intent to identify characteristics that are more representative of the observed on-site performance. Below is a list of the revised indicators. It should be noted that each of the indicators should not be viewed as a "stand alone" result, but in conjunction with all of the indicators.

The revised indicators were as follows:

- <u>Net Volume Change</u> The results of the change in net volume, as discussed above, is still considered a relevant indicator for future tests. The emphasis on the net volume change shifts to form part of a suite of indicators rather than the primary indicator of the heave test.
- <u>Penetration per Blow</u> The amount of penetration per blow provides an insight into the level of compaction of the ground; whereby a reduced amount of penetration per blow generally indicates a more-dense profile. Conversely, where the penetration per blow is high then this is indicative of relatively loose / soft ground.

Additional information can also be ascertained by observing the shape (or profile) of the graph when coupled with other indicators. Where the graph has reached a plateau, it was inferred that a degree of uniformity (and hence a consistent degree of compaction) had been achieved within the fill.

• <u>Heave per Blow</u> – The amount of heave per blow provides an indication of the suitability of the other indicators used to assess the Heave Test.

During the site works it was also observed that the amount of heave appeared to reach a limit, which is believed to be a function of the crater depth. It is postulated that when a crater exceeds a certain depth, the amount of energy being transferred up through the soil (resulting in heave) is not large enough to continue to disturb the ground surface (due to the weight and strength of the overlying material) and "register" a change in the amount of heave. Thus, any indicators that rely on only the heave (such as the net volume change) may provide a result inconsistent with the field observations.

<u>Ratio of Penetration to Heave</u> – The ratio of penetration to heave is based on the volume of the heave and the volume of the penetration (or crater) calculated per blow. The ratio of penetration to heave provides an indication of the degree of

compaction that is being achieved. When the ratio reduces to an asymptote near unity, this indicates that the limiting degree of compaction (for a given pounder weight and drop height) is being achieved. The ratio of penetration to heave needs to be viewed in conjunction with the amount of penetration per blow and heave per blow to allow that assessment to be made.

An example plot of the revised heave test indicators is presented in Figure 5-5. The net volume change (i.e. print volume – heave volume) is relatively small up to 16 blows, but then increases more rapidly with increasing number of blows. The initial penetration rate is high and initially reduces with increasing blow count. After 16 blows, the penetration per blow increases to an asymptote of what appears to be approximately 80mm per blow.

The heave per blow show the inverse of the penetration per blow; in that it increases to 0.4m^3 per blow after 16 blows and then reduces to less than 0.15m^3 per blow with increasing blow count.

The results of the ratio of penetration to heave are not too dissimilar to the net volume change, in that the ratio is relatively small up to 12 blows and then become larger with increasing number of blows (which coincides with a reduction in the amount of heave being recorded). The ratio of penetration to heave is highly variable as a standalone indicator, with the ratio of penetration to heave varying between <1 and >10 (ratio's greater than 10 have not been presented as they indicate very loose/soft materials).



Figure 5-5. Example of Revised Heave Indicators, Secondary Pass

Presented in Appendix 5A are the results of the heave tests undertaken during the dynamic compaction work. From these results, the general trends observed in the indicators are summarised as follows:

- The penetration per blow generally decreases from 200mm per blow, to the following after approximately 16 blows:
 - Typically between 80mm to 100mm per blow for the primary pass;
 - o Typically less than 80mm per blow for the secondary pass; and,
 - Typically less than 40mm per blow for the tertiary pass.
- Generally, the heave per blow shows a large, initial amount of heave that decreases with increasing blow count. Generally, the heave per blow reduces significantly after 16 blows. The initial volume of heave per blow and the subsequent reduction in heave per blow appears to be specific to the material type, moisture content and also the density.
- The ratio of penetration to heave provides an indication similar to that of the print volume minus heave volume. The ratio of penetration to heave is also highly variable and sensitive to the material type, moisture content and density.

The key advantage of using 4 indicators is that a more informed decision can be made about the ground response to dynamic compaction with increasing blow count. The net volume change, which is the only indicator of the typical heave test, is very sensitive to the amount of heave recorded during the heave test. The reliance on the net volume alone can result in an increased estimate of the number of blows required to compact the ground.

5.2.5 Extensometer Data

A review of the extensioneter data has been carried out, with the intention of correlating the results from the extensioneter data with the results of the heave testing. Example plots of the extensioneter data are presented in Figures 5-6 and 5-7, with the results of all the extensioneter data presented in Appendix 5B. The plots are presented showing the amount of settlement, where negative settlement indicates heave, in millimetres and Relative Level (R.L.) in metres.

A summary of the results of the extensometer data is as follows:

- A significant amount of heave can generally be observed in the extensometer plots, which can be more than 500mm at the ground surface. The majority of the heave occurs during the tertiary pass with twice the number of drop locations occurring within the tertiary pass than the primary and secondary pass.
- The depth to which the "zone of disturbance" extends varies from 2.5m to >5m. The "zone of disturbance" is a zone where the soil has become less dense as a result of the dynamic compaction and includes materials that may have settled due to settlement of underlying materials. This is discussed further in Chapter 6.
- The strain (the total magnitude of heave divided by the depth interval in which the zone of disturbance extends) in the upper profile typically varied between 1% and 15%. The strain is a heave strain; that is, the soil is becoming less dense.
- The depth of fill subject to settlement recorded in the extensometers extends to depths typically between 10m to 12m.
- The amount of strain (total magnitude of settlement divided by the depth over which the settlement occurs) recorded in the settlement zones varied from about 0.1% to 3%. When viewed over 1m intervals, or the distance between magnets, there are isolated areas where settlement strain was >5%. This is interpreted as a loose zone being treated during the compaction process.
- On some of the extensometer results, for example Figure 5B-14, the monitoring results show an isolated zone of no (or negative) settlement at depth. This is considered a malfunction of the instrument; in that the magnet has not been properly secured into the soil. Hence, the magnet remains stationary within the borehole while the soil surrounding the magnet settles.



Figure 5-6. Example Extensometer Results (Area 5)



Figure 5-7. Example Extensometer Results (Area 2)

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5.2.6 Comparison between Heave Test and Extensometer Data

A comparison between the results of the heave tests and the extensioneter data has been undertaken. For each heave test undertaken adjacent to an extensioneter, a plot has been developed depicting the average ground surface profile recorded during heave tests and the heave recorded by the extensioneters.

Presented on Figure 5-8 is a plot showing the distance from the centre of the drop location, on the horizontal axis, and the change in ground surface level, on the vertical axis. The ground surface level recorded from the heave test during the secondary and tertiary phases of the trial is compared to the results from the extensioneter also during the secondary and tertiary phases of dynamic compaction.



Figure 5-8. Example Plot of Comparison between Heave Test and Extensometer Test

The results generally indicate that the superposition of the heave test results gives larger heaves than the extensometer readings. This result is not unexpected as the upper magnet of the extensometer is generally 1m below the ground surface level. Additional heave could be occurring within the upper 1m resulting in greater ground disturbance, which is recorded during the heave test.

The extensometer results being recorded are assessed to be realistic and that there is no reason to believe that a defect in the instrumentation has been responsible for the heave.

5.2.7 Summary and Outcomes of Assessment of Heave Test Data

Based on the results of the review of the heave tests data and the extensometer monitoring, the following conclusions can be drawn:

- The penetration per blow for the equipment used in the study generally decreased from 200mm per blow, to the following after approximately 16 blows:
 - Typically between 80mm to 100mm per blow for the primary pass;
 - Typically less than 80mm per blow for the secondary pass; and,
 - Typically less than 40mm per blow for the tertiary pass.
- The heave per blow typically indicated initially large amounts of heave, which decreased with increasing blow count. Generally, the heave reduced significantly after 16 blows.
- The extensometer data indicated heave in excess of 500mm which can extend to depths >5m. This corresponds to a heave strain (or loosening of the soil) of >10%.

Based on a comparison between the heave test results (and superposition thereof) and the extensometer data, the magnitude and extent of the heave recorded is considered to be representative of the actual situation that is occurring on site.

Based on the results presented above, the dynamic compaction methodology could be adopted in order to:

- Reduce the degree of disturbance (both in terms of magnitude and depth) in the zone of disturbance so that it will not extend beyond 3m in depth by reducing the amount of penetration; and,
- Reduce the number of blows required so that it achieves the required performance outcomes in an efficient manner.

Based on the above results it is postulated that the dynamic compaction process delivers the majority of the effect in the initial 15 to 20 blows, with a reduced rate of improvement (below the zone of disturbance) in the remaining 10 to 15 blows. Further blows are likely to only provide minor improvement, and at the same time can have an adverse impact in increasing the depth of the zone of disturbance.

Localised areas of relatively large settlement strains (>8% in some zones, compared to an average of up to 1% strain over the entire depth profile) indicate that the dynamic compaction is compacting loose zones at depth. The overall effect of dynamic compaction appears to be small (settlement strains of 1%) but this is indicative of the condition of the ground prior to dynamic compaction – that is, highly variable with interbedded layers of loose and dense material.

It is also postulated that the 16 blows may be sufficient to compact the loose layers such that the required performance outcomes are achieved. It would also be expected that by reducing the number of blows it is likely that the degree of disturbance in the zone would also reduce.

The assessment of the above hypothesis will be the subject of the following sections.

5.2.8 Assessment of Number of Blows

In order to assess whether the majority of improvement was being delivered in the dynamic compaction in the initial 16 blows compared to the second 16 blows a full scale trial was undertaken.

Dynamic compaction was carried out in the area denoted as Area 2. Monitoring of the dynamic compaction and geotechnical investigations were undertaken after the initial 16 blows with dynamic compaction, with an additional investigation carried out after the second tranche of 16 blows (32 blows in total). The revised heave test methodology was implemented and the CPT, DHGD test and extensometer data were used to assess the dynamic compaction.

A comparison between the CPT results, DHGD test results and extensometer results is presented in the following sections.

Electric Friction Cone Penetrometer Tests

Six CPT probes were carried out after the first 16 blows of dynamic compaction works, and 5 CPT probes carried out after completion of a total of 32 blows of dynamic compaction.

The results of the CPT works at each location were collated in the following form:

• A plot of the measured cone resistance (q_c) for the first 16 blows and the second 16

blows; and,

• A plot of the ratios of the average cone resistance (q_c) and average sleeve friction (f_s), over a 1-metre interval, after 16 blows to those after 32 blows of dynamic compaction for each individual CPT.

The premise for the second plot is that when the ratio is near unity, there is limited change in the subsurface strength and density profile after the additional dynamic compaction work. Conversely, when the ratio of is high (>2), this indicates that improvement has been achieved by carrying out the additional 16 blows of dynamic compaction. Example plots of the comparison between CPT probes are presented on Figures 5-9 and 5-10.

The results of the comparison between CPT probes after 16 blows with dynamic compaction and after 32 blows with dynamic compaction are interpreted as follows:

- The ratio is mainly near unity.
- There is typically a decrease in the q_c and f_s ratios from 1.0m to 2.0m depth (R.L 23m to 22m) which is assessed as being due to the disturbance of the upper 2.0 metres during the latter stages of the dynamic compaction work works.
- Below approximately 2m local peaks and troughs in the q_c and f_s ratio (<1 or >1) are assessed as being due to variability of the soil within the fill and, in particular, the presence of gravel bands within the fill.

Downhole Gamma Density Tests and In situ Tube Sampling Results

Three DHGD tests were carried out after the completion of 16 blows of dynamic compaction and after 32 blows of dynamic compaction. In a similar manner to the CPT results, the results of the DHGD tests are presented in the following form:

- A plot of the Wet Density (t/m³) with depth recorded after 16 blows and 32 blows with dynamic compaction, averaged over a 1-metre interval; and,
- A plot of the ratio of the Wet Density between 16 blows and 32 blows with dynamic compaction, averaged over a 1-metre interval.

An example of the results of the comparison between the DHGD results after 16 blows and 32 blows with dynamic compaction results is presented in Figure 5-11.



Figure 5-9. Comparison between CPTs after 16 blows and after 32 blows



Figure 5-10. Comparison between CPTs after 16 blows and after 32 blows

The results of the comparison show that there is little change between the wet density magnitude after 16 blows and 32 blows with dynamic compaction, and the average wet density ratio generally lies close to unity.





Extensometer Monitoring

Extensometers were installed at 3 locations and were monitored during the primary, secondary and tertiary passes after 16 blows and 32 blows of dynamic compaction. The results are presented as a combined plot of the monitoring results for each pass (Primary, Secondary, and Tertiary). It should be noted that the data have been re-zeroed at the start after the first 16 blows with dynamic compaction.

Typical plots of the extensioneter monitoring results are presented in Figures 5-12 to 5-14 for the primary, secondary and tertiary passes respectively.



Figure 5-12. Extensometer Monitoring Results – Primary Pass



Figure 5-13. Extensometer Monitoring Results – Secondary Pass



Figure 5-14. Extensometer Monitoring Results – Tertiary Pass

The comparison between the extensometer results after 16 blows and 32 blows of dynamic compaction is reasonably consistent through the primary, secondary and tertiary passes at each of the 3 monitoring locations. Comparison between the results after 16 blows and 32 blows of dynamic compaction indicates that:

- Two magnets, at approximately R.L. 16.5m and 14.5m do not appear to be functioning (refer to Section 5.2.5).
- The settlement recorded at depths greater than 5m after 32 blows with dynamic compaction is approximately half of the settlement recorded after 16 blows; and,
- The amount of heave recorded is generally greater after 32 blows of dynamic compaction than after 16 blows of dynamic compaction.

Conclusions

The extensioneter monitoring results suggest that there is limited ground improvement as a result of carrying out the 32 blows of dynamic compaction, compared to 16 blows. Whilst some ground improvement is indicated by extensioneter monitoring, the majority of the improvement appears to occur within the first 16 blows.

The amount of settlement strain occurring is approximately 0.2% in both the first 16 blows and the second 16 blows. However, the biggest change in the profile appears to be the compaction of a loose zone between R.L. 12m and R.L. 13m, where a settlement strain of approximately 3% during the first 16 blows and <0.5% during the second 16 blows (refer to Figure 5-14). Based on these results it would appear that the majority of the compaction of a loose zone, at approximately 12m depth, has occurred within the first 16 blows.

The amount of heave recorded is much greater after 32 blows than after 16 blows. Therefore, by limiting the number of blows; the depth and magnitude of the heave in the upper profile will also be limited.

The CPT and DHGD data shows there is limited change in the subsurface profile between the first 16 blows and then after 32 blows. Considering that the extensometer profile indicates an average increase in density of 0.2%, it would be very difficult to identify such a change through the CPT or DHGD profile. As there are no test data showing the subsurface profile before the dynamic compaction was undertaken, the CPT and DHGD data do not show the compaction of the loose zone identified in the extensometer. More so, the CPT and DHGD results indicate that the addition of 16 blows (i.e. from 16 blows to 32 blows) does not significantly improve the ground stiffness or density.

As such it is deduced that the dynamic compaction process delivers the majority of the effect in the initial 16 blows, with a reduced rate of improvement in the remaining 16 blows. Further blows are likely to only provide minor improvement, and at the same time may have an adverse impact in increasing the depth of the zone of disturbance.

5.3 Assessment of Grid Spacing

5.3.1 Introduction

A full scale trial was undertaken to assess the effect of changing the grid spacing of dynamic compaction. For the trial, the grid spacing was varied between 4.2m to 6.0m (centre to centre). The area being subject to dynamic compaction was at least 30m x 100m, and extending up to 100m x 100m in plan area. The monitoring and testing undertaken as part of this assessment was located centrally within each area such that there was no overlap or influence from adjacent areas.

The basic premise for this assessment was to have a grid spacing with sufficient overlap of

the influence of dynamic compaction at a drop point to compact the material between the drop points. This is similar to the approach presented by Poran and Rodriguez (1992) who investigated both the vertical and lateral extent of the influence of dynamic compaction on cohesionless soils and filled ground.

The assessment of grid spacing interaction was based on a series of key monitoring indicators previously presented which were:

- Heave / Penetration Test Results; and,
- Monitoring of Extensometers.

These indicators were used to identify the degree of interaction for a range of grid spacing within different compaction areas. These tests have been previously discussed; however the following should be noted:

- The premise of assessing whether there is "interaction" between drop points is that the results of the heave / penetration tests for the tertiary pass should show a large heave volume and a subsequent small volume of the crater formed during the dynamic compaction process. Conversely, where there is no interaction, there should be relatively little heave and the amount of penetration recorded should be large.
- The extensometers should show that if there is limited interaction between drop points, the expected results from the monitoring would be either a large or small settlement with a small heave. If there is interaction between drop points, the expected results from the monitoring would be either a large or small settlement with a large heave.

5.3.2 Heave / Penetration Test Results

The results of the heave / penetration tests have been presented in Figures 5-15 and 5-16. Presented in Figure 5-15 is a plot of the penetration per blow (in m/blow); and of the heave volume per blow (in m^3 /blow). Presented in Figure 5-16 is the ratio of the print volume to the heave volume.

The results of the heave / penetration testing indicate the following:

• The average penetration per blow remains nearly constant for the different grid

spacings;

- The amount of heave volume per blow decreases with increasing grid spacing; and,
- The ratio of the penetration volume to heave volume increases with increasing grid spacing.

The results of the monitoring show a trend of decreasing interaction between drop locations with increasing grid spacing (a line of best fit is shown in Figures 5-15 and 5-16).



Figure 5-15. Results of Heave Tests – Penetration and Heave Volume per Blow



Figure 5-16. Results of Heave Tests – Ratio of Print Volume to Heave Volume

5.3.3 Extensometer Monitoring Results

The results of the extensioneter monitoring are presented in Figure 5-17 as a plot of the maximum recorded heave and settlement for each of the extensioneters recorded in areas of different grid spacing. Also presented in Figure 5-17 are lines of best fit for the heave and settlement.



Figure 5-17: Results of Extensometer Monitoring

The results of the extensometer monitoring show a consistent trend of decreasing settlement and heave with increasing grid spacing.

The amount of heave recorded for one of the extensometers installed at a 5m grid spacing shows little heave. This is consistent with a loose zone being near the ground surface and the amount of interaction between drop points being limited.

The amount of settlement recorded in the area with a 5.5m grid spacing shows a significant amount of settlement, however this is assessed to be associated with a loose zone at depth. The upper surface of the extensometer shows some heave occurring, which is consistent with the subsurface profile having interbedded loose and dense layers.

5.3.4 Summary and Conclusions

The results of the full scale trial in to the effect of the grid spacing on dynamic compaction can be summarised as follows:

- The results of the heave / penetration testing show a decreasing interaction between drop locations as the grid spacing is increased from 4.2m to 6.0m;
- The extensometer results also show a decreasing interaction between drop locations as the grid spacing increases from 4.2m to 6.0m;
- The extensioneter results showed a decrease in the depth of influence of the dynamic compaction process with increasing grid spacing.

Based on the results of the trial, and the monitoring it was concluded that a grid spacing of 4.5m had interaction between drop locations and was adopted for the works.

5.4 Use of Topographic Survey

5.4.1 General

An assessment was carried out regarding the use of the topographic survey to record the ground surface response at each drop location in the form of a heave volume and print volume. The topographic survey information was supplemented by the results of the heave tests carried out during the monitoring of the dynamic compaction.

5.4.2 Topographic Survey Process

The topographic survey consisted of recording 3 to 4 survey strings (a continuous line of survey), which used 8 or more points at approximately the same R.L., at each drop location. The survey strings were collected at the base of the crater, top of the crater edge and at the visible lateral extent of the heave profile. A survey string was also recorded between the top of the crater and the extent of the profile.

A schematic diagram showing survey strings, indicative survey points in plan and typical cross section of print and heave profile at a drop location is presented in Figure 5-18.

The recorded data were used to calculate heave volume and print volume and hence ratio of print volume to heave volume at each drop location.





5.4.3 Topographic Survey Results

The assessment was based on topographic survey results recorded after each of the interlocking primary, secondary and tertiary passes. The topographic survey results are presented in the form of contour plots as follows:

- The heave volume and print volume recorded after the primary and secondary passes at each drop location;
- The ratio of print volume to heave volume (RPH) recorded after the primary and secondary passes;

- The heave volume and print volume recorded after the tertiary pass; and,
- The RPH recorded after the tertiary pass.

Example plots are presented in Figures 5-19 to 5-22.

5.4.4 Assessment of Topographic Survey Results

The premise for this assessment was that when the RPH was low (<2) the ground was being displaced rather than compacted, indicating that the ground was sufficiently dense that it reflected the energy delivered by the dynamic compaction process. Conversely, when the RPH was large, the ground was being compacted.

It should be noted that the delineation of a "low" RPH to a "high" RPH was based on site specific experiences such as the results from the other key indicators and the engineering properties of the subsurface profile.

A summary of the results is as follows:

- There was a significant increase of the area with 2< RPH <4 and a significant reduction of area with a RPH >4.
- The print volume remained consistent.
- The heave volume generally increased.

In areas that exhibited RPH <4 after the primary and secondary passes, the RPH increased as a result of the tertiary pass. Conversely, in areas that exhibited RPH >4 after the primary and secondary passes, RPH decreased as a result of the tertiary pass.

Based on the above, it was inferred that the increase in RPH after the tertiary pass relative to the results after the primary and secondary passes was a result of the disturbance of the near surface material rather than compacting the fill at depth. The results of the extensometer monitoring, which are discussed further in Section 5.4.5, show that the amount of heave increased significantly during the tertiary pass. While some settlements were recorded, these were <50mm (compared with the heave which was >600mm).



Figure 5-19. Plot of Print and Heave Volume after Primary and Secondary Pass



Print Volume to Heave Volume Ratio

Figure 5-20. Plot of Print to Heave Ratio After Primary and Secondary Pass


Figure 5-21. Plot of Print and Heave Volume after Tertiary Pass



Print Volume to Heave Volume Ratio

Figure 5-22. Plot of Print to Heave Ratio After Tertiary Pass

The decrease in RPH recorded in the survey after the tertiary pass relative to the results after the primary and secondary passes was inferred to be due to the ground becoming compacted through the application of the dynamic compaction (even though the near surface material has been disturbed).

Hence, in areas that have RPH <2 after the primary and secondary passes, the ground appeared to be at a relatively high density prior to the commencement of dynamic compaction. During the tertiary pass, it appeared that the near-surface material became disturbed as a result of the dynamic compaction.

5.4.5 Results of Extensometer Monitoring

The results of the extensioneter monitoring are presented in Figures 5-23 and 5-24.

The results of the extensometer monitoring show that there is a significant amount of heave (or negative settlement) recorded in the extensometers, particularly between the secondary and tertiary passes. For example, in Figure 5-24, an increase in heave of over 300mm was recorded between the secondary and tertiary pass, whereas only 100mm of heave was recorded between the primary and secondary pass.

The extensometer results are broadly consistent with the observations made in the survey results; in that the amount of near surface ground disturbance in the primary and secondary pass is not as much as that recorded in the tertiary pass. In the context of the usefulness of carrying out the survey after the tertiary pass, the extensometer data confirms the hypothesis presented above that the ground becomes too disturbed during the tertiary pass of dynamic compaction to enable meaningful data to be obtained from carrying out the survey. As such, there appears to be no benefit in conducting a topographic survey after the tertiary pass of dynamic compaction.



Figure 5-23. Results of Extensometer Monitoring



Figure 5-24. Results of Extensometer Monitoring

5.4.6 Results of CPT Probes

For this assessment, the CPT data have been plotted as a ratio of the tip resistance, q_c , and skin friction, f_s , in an area that has a low ratio of penetration volume to heave volume to the results of a CPT probe carried out in an area that has a (relatively) high ratio of penetration volume to heave volume.

The objective of this comparison is to investigate whether the CPT results in an area with a

low ratio of penetration volume to heave volume are typically of higher magnitude than the CPT results in an area with a high ratio of penetration volume to heave volume.

An example of this comparison is presented in Figure 5-24.





As can be seen from Figure 5-25, the results are typically above unity to a depth of approximately 10m; except in two distinct intervals: between 2m and 3m and also between 5.5m and 6.5m. Review of the original CPT results indicate some near surface disturbance within the upper 3m which has distorted the ratio presented in Figure 5-25; and also the presence of a localised peak in the results of the CPT performed in area of high penetration volume to heave volume, synonymous with intersecting gravel. This is also indicative of

the reason that the q_c ratio is significantly different to the f_s ratio at this depth.

The results of the CPT being above unity, indicates that the soil generally has a higher cone tip resistance, q_c , in the areas having a low penetration volume to heave volume ratio when compared to those areas that have a high penetration volume to heave volume ratio.

5.4.7 Summary of Assessment

The results of the topographic survey were used to assist in the assessment of uncontrolled fill that was remediated with dynamic compaction by providing a spatial plot showing areas of relative density (for a consistent fill type).

The assessment carried out indicated that the topographic survey should only be carried out after the completion of the primary and secondary passes of the dynamic compaction process. Due to the amount of disturbance of the upper profile, it is considered of little additional benefit to carry out the topographic survey after the tertiary pass of dynamic compaction.

The results of the extensioneter monitoring confirmed this hypothesis, whereby that the upper zone was disturbed during the dynamic compaction process; with most of the disturbance occurring during the tertiary pass of dynamic compaction.

The spatial plot of the RPH after the primary and secondary passes could be used to assist in the location of the geotechnical investigations after dynamic compaction has been carried out. For example, the investigation locations could target areas where the RPH is high.

5.5 Assessment of Landform

As a result of the refinement of the dynamic compaction process, the landform was compacted using dynamic compaction. Upon completion of the dynamic compaction process, analyses were carried out to assess the performance of the landform post dynamic compaction.

Similar to the results of the geotechnical investigations presented in Chapter 3, summary plots are presented in Appendix 5C detailing the results of the geotechnical testing carried out at each investigation station. Presented in the summary plots is the following information:

• A graphic log showing the ground surface profile;

- Graph 1 The cone tip resistance (q_c) from the CPT;
- Graph 2 The wet density from the down-hole gamma density test and the *in situ* density testing carried out from tube samples;
- Graph 3 The caliper width measured by the down-hole gamma density tool;
- Graph 4 The dilatometer modulus calculated from each test; and,
- Graph 5 The results of the extensioneter monitoring.

Using the methodology described in Chapter 4, analyses have been undertaken to assess the likely settlement and slope of the landform post-dynamic compaction, including the effect on buildings (in terms of building strain).

The results of the assessment are presented in Table 5-1. Also included in Table 5-1 is an extract of the pre-dynamic compaction results presented in Table 4-2.

A comparison of the results indicates the following:

- The average maximum sagging or hogging slope is larger for the post-dynamic compaction landform, compared to the pre-dynamic compaction landform. However the pre-dynamic compaction landform has a significantly higher maximum and standard deviation in the performance.
- The average building strain recorded post-dynamic compaction is within the recommended strain for a Damage Category of 0. The maximum building strain calculated (for a single storey) is just outside the Damage Category 0 criteria, with a recorded strain of 0.05%.
- There is a significant reduction in the maximum building strain calculated for the landform post-dynamic compaction, when compared to the pre-dynamic compaction landform. The maximum building strain calculated is almost halved.
- There is also a significant reduction in the standard deviation of the post dynamic compaction assessment categories, compared to the pre-dynamic compaction results.

Table 5-1. Assessment of Landform

Assessment Category		Pre-Dynamic Compaction				Post Dynamic Compaction			
		Count	Maximum	Average	Std Dev	Count	Maximum	Average	Std Dev
Slope Induced by		9	1 in 1,755	12,146	11,668	24	1 in 3,249	1 in 9,130	5,096
Settlement									
Strain	Single	9	0.103%	0.038%	0.036%	24	0.0582%	0.0255%	0.0130%
induced by	Storey								
Settlement	Double	9	0.121%	0.048%	0.040%	24	0.0710%	0.0329%	0.0173%
	Storey								

The results presented in Table 5-1 show a significant improvement in the likely performance of the landform, post-dynamic compaction. In particular, there is a large decrease in the variation of the results. This is indicative of applying a consistent level of energy across the site to compact the area and thus providing a more consistent profile.

Furthermore, the results of the assessment indicate that the maximum recorded building strain is within the criteria noted in Chapter 4 for an acceptable building performance (unlike the pre-dynamic compaction assessment).

5.6 Summary and Conclusions

This chapter presented the results of a full scale trial of dynamic compaction and focused on presenting the results of a detailed assessment of the number of blows undertaken at each drop location and the effect of changing the grid spacing between drop locations. Studies in to these particular areas have not been undertaken previously.

The results of the full scale trial showed that, for a site containing uncontrolled fill with a highly variable density, the extensometers were the better method for assessing the effect of dynamic compaction. The amount of settlement strains recorded varied between <0.1% through to approximately 3% over the depth of compaction. However, what became more apparent is that the dynamic compaction process induced settlement strains up to 8% in isolated loose zones, even to depth in the order of 12m.

Further analyses have been undertaken to assess the likely performance of the landform, including building strain, post dynamic compaction. The results are compared to the predynamic compaction assessment presented in Chapter 4, and show that with the compaction of isolated loose that were present in the results presented in Chapter 4 the expected performance and consistency across the site is calculated to be at an acceptable level (Damage Class 1 or less).

The results of the dynamic compaction work are discussed in more detail in Chapter 6, focusing on the behaviour in the zone where heave occurs; and also the results of monitoring in the zone where settlement occurs. These zones are referred to within this chapter, and also in literature, but are the focus of Chapter 6.

APPENDIX 5A

RESULTS OF HEAVE / PENETRATION TEST



Figure 5A – 1. Heave Test Results, Area 1 Primary Pass





Figure 5A – 2. Heave Test Results, Area 1 Primary Pass





Penetration per Blow



Heave Per Blow





Figure 5A – 3. Heave Test Results, Area 1 Secondary Pass





Figure 5A – 4. Heave Test Results, Area 1 Tertiary Pass



No of Blows

Figure 5A – 5. Heave Test Results, Area 1 Tertiary Pass



Figure 5A – 6. Heave Test Results, Area 4 Primary Pass







Figure 5A – 7. Heave Test Results, Area 4 Primary Pass





Figure 5A – 8. Heave Test Results, Area 4 Secondary Pass



Figure 5A – 9. Heave Test Results, Area 4 Secondary Pass





Figure 5A – 10. Heave Test Results, Area 4 Tertiary Pass



Penetration per Blow



Heave per Blow





Figure 5A – 11. Heave Test Results, Area 4 Tertiary Pass



Figure 5A – 12. Heave Test Results, Area 3 Primary Pass





Figure 5A – 13. Heave Test Results, Area 3 Primary Pass

-10.0

Heave Penetration Test



Figure 5A – 14. Heave Test Results, Area 3 Secondary Pass

No of Blows



Figure 5A – 15. Heave Test Results, Area 3 Secondary Pass





Figure 5A – 16. Heave Test Results, Area 3 Tertiary Pass





Penetration per Blow











Figure 5A – 17. Heave Test Results, Area 3 Tertiary Pass



Figure 5A – 18. Heave Test Results, Area 3 Tertiary Pass



Figure 5A – 19. Heave Test Results, Area 3 Tertiary Pass









Figure 5A – 20. Heave Test Results, Area 5 Primary Pass







Figure 5A – 21. Heave Test Results, Area 5 Primary Pass



Figure 5A – 22. Heave Test Results, Area 5 Secondary Pass











Figure 5A – 23. Heave Test Results, Area 5 Secondary Pass











Figure 5A – 24. Heave Test Results, Area 5 Tertiary Pass





Figure 5A – 25. Heave Test Results, Area 5 Tertiary Pass


Figure 5A – 26. Heave Test Results, Area 5 Tertiary Pass



No of Blows

Figure 5A – 27. Heave Test Results, Area 5 Tertiary Pass





Figure 5A – 28. Heave Test Results, Area 8 Primary Pass



Figure 5A – 29. Heave Test Results, Area 8 Primary Pass











No of Blows

Figure 5A – 30. Heave Test Results, Area 8 Secondary Pass





Figure 5A – 31. Heave Test Results, Area 8 Tertiary Pass



Figure 5A – 32. Heave Test Results, Area 8 Tertiary Pass



Heave Penetration Test



Figure 5A – 33. Heave Test Results, Area 8 Primary Pass











Figure 5A – 34. Heave Test Results, Area 8 Secondary Pass



Figure 5A – 35. Heave Test Results, Area 8 Tertiary Pass





Penetration per Blow







Ratio of Penetration to Heave

Figure 5A – 36. Heave Test Results, Area 8 Primary Pass



Figure 5A – 37. Heave Test Results, Area 8 Secondary Pass

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Heave Penetration Test
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Figure 5A – 38. Heave Test Results, Area 8 Primary Pass

Heave Penetration Test



Figure 5A – 39. Heave Test Results, Area 8 Secondary Pass



Figure 5A – 40. Heave Test Results, Area 6 Tertiary Pass

Heave Penetration Test



Figure 5A – 41. Heave Test Results, Area 9 Primary Pass





Figure 5A – 42. Heave Test Results, Area 9 Primary Pass



Figure 5A – 43. Heave Test Results, Area 9 Primary Pass



Figure 5A – 44. Heave Test Results, Area 9 Secondary Pass



Figure 5A – 45. Heave Test Results, Area 9 Tertiary Pass





Figure 5A – 46. Heave Test Results, Area 9 Tertiary Pass







Figure 5A – 47. Heave Test Results, Area 9 Primary Pass



Figure 5A – 48. Heave Test Results, Area 9 Secondary Pass



Figure 5A – 49. Heave Test Results, Area 9 Tertiary Pass



Figure 5A – 50. Heave Test Results, Area 9 Tertiary Pass



Figure 5A – 51. Heave Test Results, Area 9 Primary Pass



Figure 5A – 52. Heave Test Results, Area 9 Secondary Pass





Figure 5A – 53. Heave Test Results, Area 9 Tertiary Pass



Figure 5A – 54. Heave Test Results, Area 9 Tertiary Pass

APPENDIX 5B

RESULTS OF EXTENSOMETER MONITORING



Figure 5B – 1. Extensometer Results, Area 1



Figure 5B – 2. Extensometer Results, Area 1



Figure 5B – 3. Extensometer Results, Area 2



Figure 5B – 4. Extensometer Results, Area 2



Figure 5B – 5. Extensometer Results, Area 4



Figure 5B – 6. Extensometer Results, Area 4


Figure 5B – 7. Extensometer Results, Area 4



Figure 5B – 8. Extensometer Results, Area 4



Figure 5B – 9. Extensometer Results, Area 5



Figure 5B – 10. Extensometer Results, Area 5



Figure 5B – 11. Extensometer Results, Area 5



Figure 5B – 12. Extensometer Results, Area 5



Figure 5B – 13. Extensometer Results, Area 9



Figure 5B – 14. Extensometer Results, Area 9



Figure 5B – 15. Extensometer Results, Area 9

APPENDIX 5C

RESULTS OF GEOTECHNICAL INVESTIGATIONS



Figure 5C – 1. Area 1, Investigation Station 1



Figure 5C – 1. Area 1, Investigation Station 2



Figure 5C – 1. Area 2, Investigation Station 1



Figure 5C – 1. Area 2, Investigation Station 2



Figure 5C – 1. Area 2, Investigation Station 3



Figure 5C – 1. Area 2, Investigation Station 4



Figure 5C – 1. Area 3, Investigation Station 1



Figure 5C – 1. Area 3, Investigation Station 2



Figure 5C – 1. Area 4, Investigation Station 1



Figure 5C – 1. Area 4, Investigation Station 2



Figure 5C – 1. Area 4, Investigation Station 3



Figure 5C – 1. Area 4, Investigation Station 4



Figure 5C – 1. Area 5, Investigation Station 1



Figure 5C – 1. Area 5, Investigation Station 2



Figure 5C – 1. Area 5, Investigation Station 3



Figure 5C – 1. Area 5, Investigation Station 4



Figure 5C – 1. Area 6, Investigation Station 1



Figure 5C – 1. Area 6, Investigation Station 2



Figure 5C – 1. Area 6, Investigation Station 3



Figure 5C – 1. Area 7, Investigation Station 1



Figure 5C – 1. Area 7, Investigation Station 2



Figure 5C – 1. Area 7, Investigation Station 3



Figure 5C – 1. Area 7, Investigation Station 4



Figure 5C – 1. Area 8, Investigation Station 1



Figure 5C – 1. Area 8, Investigation Station 2



Figure 5C – 1. Area 8, Investigation Station 3


Figure 5C – 1. Area 8, Investigation Station 4



Figure 5C – 1. Area 9, Investigation Station 1



Figure 5C – 1. Area 9, Investigation Station 2



Figure 5C – 1. Area 9, Investigation Station 3



Figure 5C – 1. Area 9, Investigation Station 4

CHAPTER SIX

REVIEW OF DYNAMIC COMPACTION

6 REVIEW OF DYNAMIC COMPACTION

6.1 General

Based on the results of the trials, the review of the effect of dynamic compaction has been undertaken in three zones, termed the Heave Zone; the Settlement Zone and an Undisturbed Zone. This approach is different to that of Smits and De Quelerij (1989) who presented a model of three zones, namely a Compacted Zone, an Elastically Deformed Zone and an Undisturbed Zone.

Presented in Figure 6-1 is a copy of the Smits and De Quelerij (1989) model and that considered in this chapter.







Figure 6-1. Representation of effect of Dynamic Compaction

The difference in the approach is that the Smits and De Quelerij (1989) model only focuses on the compaction that results from the dynamic compaction process, whereas the alternative model considers both the disturbance (or heave) and the settlement that results from dynamic compaction.

The results of the field trials presented in Chapter 5 indicate that a significant amount of heave occurs around the base of the pounder (the Heave Zone), extending to depths in the order of 3m to 5m; overlying a zone that has compressed (the Settlement Zone). The Settlement Zone extends to depths in the order of 12m, below which the subsurface profile remains undisturbed, where the dynamic compaction has no effect, overlying rock at depths of between 12m and 14m.

This chapter examines the Heave and Settlement zones and what occurs within these zones during the dynamic compaction process.

In Section 6.2, the effect of dynamic compaction in the Heave Zone is analysed. The Heave Zone becomes an important issue, particularly in the context of using dynamic compaction for ground treatment of residential developments. Residential developments typically have shallow footings, with the zone of influence underneath the footing extending typically to 3m. In order to review the Heave Zone in more detail, Section 6.2 initially focuses on the surface characterisation of the Heave Zone and then results are presented of the effect of dynamic compaction in this zone. As noted in Chapter 2, there is limited published information on the Heave Zone, hence comparisons to previous work have not been undertaken but a comparison to bearing capacity theory is presented.

In Section 6.3, the effect of dynamic compaction in the Settlement Zone is analysed. The Settlement Zone is also an important issue, as this is generally the intent of the dynamic compaction process - i.e. to compact the ground. Section 6.3 focuses on assessing the depth of the Settlement Zone and the magnitude of the settlement induced. The results of the trial are compared with the literature and case studies, and refinement of previous correlations is provided.

6.2 Review of Heave Zone

6.2.1 General

Based on the results of the trials, it appears that the Heave Zone is undergoing a bearing capacity failure, and as a result deforms the ground surface and becomes less dense. In order to test this hypothesis, an initial review of bearing capacity theory is presented in Section 6.2.2 with particular emphasis on quantifying the following:

- The shape of the bearing capacity failure envelope;
- The lateral extent of the bearing capacity failure envelope; and,
- The depth of bearing capacity failure envelope.

The subsequent section shows in more detail some of the results of the trial – in particular the surface characterisation of the heave profile and its lateral extent. These results are compared to the bearing capacity theory and initial correlations are presented.

The results of the trial are also used to assess the depth of the Heave Zone, in particular looking at the comparison of the extensioneter results to the depth of the print from dynamic compaction. Again, these results are also compared to the literature on bearing capacity such that correlations can be developed for predicting the depth of the Heave

Zone.

Finally, in Section 6.2.5, results are presented from the field trial quantifying the effect of dynamic compaction in the Heave Zone.

6.2.2 Background

The Literature Review presented in Chapter 2 presents detailed analysis and modelling of the Settlement Zone; however the analysis and modelling in the upper Heave Zone has not been undertaken to the same extent. Case studies on dynamic compaction note that the Heave Zone is treated by a tamping pass (for example Dumas *et al.* (1994)) but little information is presented quantifying the effect of the tamping pass. Furthermore, geotechnical testing such as the SPT and pressuremeter, for example, are typically not carried out until depths of 2m. This could potentially miss assessing whether the near surface materials have been treated by the dynamic compaction process.

When dynamic compaction work is undertaken for housing developments, the upper 3m has a significant effect on the performance of the footings. As such, the quantification and understanding of the behaviour in the Heave Zone becomes an important issue – and the focus of Section 6.3.

In order to quantify and understand the behaviour in the Heave Zone, bearing capacity theory has been used to model the Heave Zone; in particular, the shape, depth and lateral extent of the affected area.

Vesic (1973) presented a review paper incorporating major contributions on bearing capacity theory at the time of issue of the paper. Generally, there are three modes of failure of a foundation subject to load, namely:

- General Shear Failure: When general shear failure occurs, there is a well-defined shear surface from one edge of the footing to the ground surface level. Unless the footing is constrained, a general shear failure usually results in a rotation or tilting of the footings. There is also a bulging of the soil adjacent to the footing on both sides. General Shear Failure is generally associated with failure occurring within dense (or stiff) soils.
- **Punching Shear Failure:** Punching failure does not have a well-defined failure surface, more-so there is a vertical shear adjacent to the footing resulting from soil compressing underneath the footing. Unlike a general shear failure, in a punching

shear failure the soil surrounding the footing remains relatively unchanged. Punching failures generally occur in loose (or soft) soil profiles.

• Local Shear Failure: Local shear failure occurs immediately adjacent to a footing, and consists of a wedge and slip failure extending from the edge of the footing to the ground surface level. Vertical settlement of the footing is also observed, however the shear surfaces terminate within the soil mass. Typically, there is not significant tilting of the footing, as is typical with General Shear Failure, but there is a significant amount of vertical movement.

The modes of failure presented by Vesic (1973) are illustrated in Figure 6-2.



- (a) General Shear
- (b) Punching Shear

(c) Local Shear

Figure 6-2. Modes of Bearing Capacity Failure (Vesic, 1973)

The shear surface in a General Shear Failure case for a footing of width B and length L has been developed by Vesic (1973) using the following simplifications:

- The shearing resistance of the overburden is neglected (between section bc in Figure 6-3, below);
- The friction between the soil and the footing plus the friction between the founding soil and overburden is neglected (between section ad and ab in Figure 6-3, below);
- The length of the footing, L, is assumed to be large compared to the width, B.

The footing is also assumed to be founded in a homogeneous soil with a unit weight of γ , and strength characteristics defined by a cohesion, c, and friction angle, ϕ .

These simplifications are shown in Figure 6-3.



Figure 6-3. Definition of Bearing Capacity Problem (from Vesic, 1973)

Vesic (1973) divided the shear surface into three zones, as shown in Figure 6-4. Zone I is an active Rankine zone, which pushes the Prandtl zone (Zone II) sideways into the passive Rankine zone (Zone III) which displaces sideways and in an upwards direction.



Figure 6-4. Simplification of the Shear Surface (Vesic, 1973)

The shear surface is defined by *ACDE*, in Figure 6-4, and contains two straight lines (sections *AC* and *DE*) inclined at $45^{\circ} + \frac{\phi}{2}$ and $45^{\circ} - \frac{\phi}{2}$ to the horizontal. The shape of the curve between CD depends on the internal angle of friction, ϕ , and the ratio $\gamma B/q$ noting the following:

- If it is assumed that γ = 0 (i.e. weightless soil) then the curve between CD is a circle;
- Where the friction angle, φ, reduces to 0 then the curve between CD is also a circle; and,
- Where φ > 0, then the curve between CD lies between a logarithmic spiral and a circle.

These findings have been confirmed by De Beer and Vesic (1958).

The shear surface for static and dynamic loading on small footings was also investigated by Selig and McKee (1961), and of particular relevance in their work was mapping the shear surface both in plan and section (subsurface). Presented in Figure 6-5 are plan views of the failure pattern for a circular and square footing. These views bear the resemblance of the results of the heave tests presented in Chapter 5 of this dissertation.



Figure 6-5. Failure Pattern for a Circular footing and Square Footing (Selig and McKee, 1961)

Selig and McKee (1961) also presented photographs of a symmetrical failure of a footing under static and dynamic loading. The resultant shear surfaces are presented in Figure 6-6 and are similar to that noted in Vesic (1973).



(a) Circular Footing





Figure 6-6. Footing after Failure (Selig and McKee, 1961)

These failure surfaces compare well to that noted in Vesic (1973) and also the experimental results achieved by De Beer and Vesic (1958).

In addition to defining the failure surface through mapping the shear surface, Meyerhof (1948) undertook experiments into bearing capacity and developed plots of the maximum failure surface width, at the ground surface level, to the footing width against the length/width ratio of the footing. Meyerhof (1948) developed different plots for different footing depths, a copy of which is presented in Figure 6-8, noting that as the length of the footing increases the width of the failure surface approaches an asymptote.



Figure 6-8. Relationship of Failure Surface Width and Length of Footing (Meyerhof, 1948)

Chen (1975) presented a more simplistic approximation for the shear surface underneath a footing during bearing capacity failure, equating the shear surface to a circle extending between the footing edge and the adjacent ground surface level, as shown on Figure 6-9.



Figure 6-9. Simplification of Shear Surface (Chen, 1975)

While this solution provides a simplistic approach for bearing capacity the oversimplification results in a significantly shorter – and shallower – extent of the shear surface when compared to the more traditional approach noted in Vesic (1973).

Recent work has also been carried out using Finite Element Analysis to model bearing capacity failures. An example of which is the work undertaken by Merifield *et al.* (1995). Merifield *et al.* (1995) carried out numerical analysis to assess the bearing capacity of a rigid footing on a two-layer clay model. While this paper is primarily focused on the analytical aspects of modelling the bearing capacity behaviour, Merifield *et al.* (1995) presented the results of modelling a bearing capacity failure in homogenous (clay) soil. A copy of the velocity diagram, representing the movement of the soil, and the general shear failure envelope is presented in Figure 6-10.



Figure 6-10. Velocity Diagram for a Homogeneous soil Profile (from Merifield *et al*, 1995)

The results presented by Merrifield *et al.* (1999) were comparable with those of Vesic (1973) for the homogenous soil case. However, for a two layered clay of differing strength, the failure envelope varied significantly depending on the ratio of undrained shear strength of the upper layer to the lower layer. The results of the modelling work by Merifield *et al.* (1999) indicated that that for stiff clays overlying soft clays there are a number of different mechanisms involved in bearing capacity failure mode. A corollary of their work is that the use of classical theory to model the shape of the failure surface becomes unreliable when multiple failure mechanisms occur (i.e. the soil is not homogenous).

In this study, the failure surface under a bearing capacity failure can be defined by the shape as shown in Figure 6-4, above. Notwithstanding the findings of Merrifield *et al.* (1999), the original homogenous soil model is considered as an initial, simplistic case.

For simplicity it is assumed that the density of the soil is 0; the depth and lateral extent of the shear surface can be readily defined as a function of the friction angle, ϕ , and the depth of foundation. The characterisation of the site materials (presented in Chapter 3) indicates that the friction angle of the materials encountered ranges between 24° to 30°. Therefore, the maximum depth of the shear surface can be calculated to occur midway between points CD on the failure surface defined in Figure 6-4 and, using basic geometry, is between 1.15m and 1.13m below the footing (or the pounder in the case of dynamic compaction) for an assumed footing width of 1.8m.

The lateral extent of the shear surface is defined by the embedment of the footings however can be estimated by Vesic (1973) as shown in Equation [6-1]:

$$x = B + \left(\frac{\frac{B}{2}\tan\left(45 + \frac{\phi}{2}\right) + D}{\tan\left(45 - \frac{\phi}{2}\right)}\right)$$
[6-1]

Where x = lateral extent of bearing capacity failure;

 ϕ = friction angle;

B = width of footing (or pounder in the case of dynamic compaction) and,

D = Embedment depth of the footing (or pounder in the case of dynamic compaction).

Presented in Figure 6-11 is a plot showing the effect of embedment on both the depth and the lateral extent of the shear surface (or Heave Zone, in the case of dynamic compaction) for a range of assumed friction angles of the soil.

It should be noted, that even though bearing capacity theory provides an initial guidance for assessing the depth and lateral extent of the shear failure; there is no published information available regarding the change in the soil within the zone undergoing bearing capacity failure (or the Heave Zone, in the case of dynamic compaction).



Figure 6-11. Plot showing Effect of Embedment on Depth and Lateral Extent of Shear Surface

It should be noted that the research presented, and subsequently used, has been based on the development of a shear failure surface under a static loading (not dynamic loading). The impact of the different loading conditions has not been investigated as part of this study, but could be considered as part of future works.

6.2.3 Surface Characterisation and Lateral Extent of Heave Zone

An initial review was carried out to characterise the Heave Zone based on the ground response at the surface. Three characteristic patterns of surface heave became evident during the dynamic compaction process, namely: No Heave; Narrow and Concentrated Heave; and Wide Heave.

Sketches and photographs showing these different types of heave are shown in Figures 6-12 to 6-14.

These three characteristic types of observed heave are comparable to the three modes of failure for bearing capacity, as described by Vesic (1973) in Figure 6-2. That is, the No Heave is indicative of the punching shear failure; the Narrow and Concentrated Heave is indicative of local shear failure and the Wide Heave is indicative of the general shear failure.







Figure 6-13. Narrow and Concentrated Heave





Figure 6-14. Wide Heave

The Heave / Penetration tests were used to quantify the shape of the heave profile from different tests. Presented in Figure 6-15 is an example of the heave profile recorded.



Figure 6-15. Heave Profile Recorded

The results of the heave / penetration tests show the following:

- The Heave Zone generally extends between 5m and 9m from the centre of the crater.
- There were no areas where the ground surface profile showed "No Heave".
- Where the ground surface profile is "Narrow and Concentrated" (refer to Figure 6-13), the lateral extent of the heave was recorded to be 5m at every location.
- Where the ground surface profile is "Wide" (refer to Figure 6-14), the lateral extent of the heave varied between 5m and 9m. The variation of the lateral extent of the heave in the Wide Heave profile was related to the maximum heave recorded; that

is where the maximum amount of heave recorded was large, the lateral extent was generally large as well.

It should be noted that the above results are based on a relatively small dataset (15 points), with the depth of crater ranging from 1.1 to 2.6m. However the results appear to be somewhat dependent upon the depth of the crater, i.e. for a small amount of penetration the lateral extent of the heave zone also appears to be small.

The results of the heave / penetration tests can be compared to the extent of the shear zone estimated using bearing capacity theory. Typically, the embedment of the pounder ranged from 2m to 3m and the friction angle of the soil can be assumed to be 28° ; hence there is good correlation with the results presented in Figure 6-11.

It should be noted, however, that the bearing capacity theory does infer that the soil failure mode is a general shear failure. Therefore, the inherent assumption is that the soil mass in Zone III in Figure 6-4 (in the passive Rankine zone) remains intact during the shearing process. Where dynamic compaction is undertaken in very loose soils, the mode of failure is initially punching shear, prior to forming a general shear surface. The lateral extent of the Heave Zone shown on Figure 6-11 is generally only applicable for cases where the soil is failing in general shear or local shear, but this simple approach also appears to provide a reasonable assessment of the region of soil affected in looser soils, too.

6.2.4 Depth of Heave Zone

The depth of the Heave Zone was recorded during the trial using the extensioneter monitoring data, the heave / penetration tests and also the depth of the crater at each drop location. The location of each of these tests, relative to the drop location, is shown in Figure 6-16.

Initially a comparison was undertaken between the depth of the crater from the heave / penetration test and the depth of the heave zone assessed from the extensioneter data. The results of the comparison are presented in Figure 6-17, below.

These results show that the depth of Shear Surface (or Heave) Zone is typically about 2.5 times that of the depth of the crater resulting from the dynamic compaction process. Therefore, for a crater that extends to 2m, the depth of the Heave Zone typically extends to approximately 5.0m.



Figure 6-16. Location of Testing

Considering the location of the extensometer, i.e. approximately 2.25m from the centreline of the crater, the depth of the shear surface generated from the dynamic compaction process extends a lot deeper than would be suggested by bearing capacity theory for a homogenous soil. From bearing capacity theory, the depth of the shear zone is approximately half the width of the footing – or in the case of dynamic compaction, half the width of the pounder (0.9m). By contrast, a comparison of the results of the depth of the Heave Zone to the bearing capacity theory shows that the depth of the Heave Zone is about 5 times that estimated by bearing capacity theory.

The results of the numerical modelling presented by Merifield *et al.* (1995) indicate that, for a layered soil profile, the mechanism of the shear failure is a combination of punching shear and general shear for a stiff clay overlying a soft clay. This appears to be consistent with what is observed during the dynamic compaction process. Initially there is a punching type failure, resulting in a narrow and deep shear surface profile well below the

depth of the crater.



Figure 6-17. Comparison of Depth of Heave Zone

There is also general shear occurring, resulting in the lateral extent of the Heave Zone to be reasonably consistent with bearing capacity theory for a homogenous soil. As the depth of the shear surface is considerably deeper than that estimated by bearing capacity theory, the angle at which the shear surface intersects the ground surface would also be steeper than that assumed by bearing capacity theory.

Assuming that the results of the extensioneter monitoring is the deepest location of the shear surface, then the extent of the shear surface profile can be back-analysed using the field data. With reference to Figure 6-4 and the bearing capacity theory presented in Section 6.2, the following can roughly be inferred:

- Due to the significant amount of disturbance at the shear surface the friction angle of the material at the shear interface approaches zero. Therefore, the straight line *AC* in Figure 6-4 extends from the centre of the crater at an angle of 45° (instead of 45° + φ/2);
- The other straight line segment DE, in Figure 6-4, also extends at an angle of 45° (instead of 45° + φ/2) as φ reduces to zero;
- As a result, the points C and D in Figure 6-4 become superimposed.

This simplification of bearing capacity theory and geometry would need to be further considered based on numerical modelling and mapping of the shear surface occurring during dynamic compaction; however it does provide an indication of the extent of the Heave Zone.

The depth of the Heave Zone below the crater depth has also been assessed as a function of the depth of the crater. As can be seen from Figure 6-18, there is a large amount of scatter, however the depth that the Heave Zone extends below the crater appears to diminish with increasing crater depth.



Figure 6-18. Depth of Heave Zone beyond Crater

This is somewhat contradictory to bearing capacity theory; however, consideration needs to be given to the soil conditions that would result in a crater depth beyond 2.5m. That is, soil that has a relatively shallow crater depth during dynamic compaction would typically be dense, and therefore have a Heave Zone that extends well beyond the crater depth. Conversely, soil conditions that results in deep craters during dynamic compaction are typically loose, and therefore would have a Heave Zone that doesn't extend much further beyond the depth of the crater.

Consideration would also need to be given to the stress levels within the soil, and also the near surface soil becoming loosened by dynamic compaction. The results presented in Chapter 5 (and discussed in more detailed in Section 6.2.5) show that heave strains could

be in the order of 10%. Strains of this magnitude would need to be considered when estimating the depth and extent of the Heave Zone.

In summary, the results of the study show that the depth to which dynamic compaction loosens the soil is much deeper than the depth of the craters that are formed. This is a result of the bearing capacity-type failure mechanism that occurs at the base of the pounder, causing the heave of the ground surface profile. When heave of the ground surface occurs, this indicates that the Heave Zone typically extends to depths approximately 1.9 times that of the depth of the craters.

6.2.5 Effect within Heave Zone

The results of the extensometer monitoring have been used to calculate the average strain over the depth of the Heave Zone. From the results of the extensometer monitoring, the average heave strain in areas subject to 16 blows is 5.2%. The average strain increases to 8.1% in an area subject to 32 blows, which equates to a 2.9% increase in heave strain.

These results show that, firstly, the strain within the Heave Zone is extremely high (over 5%) which would then need to be treated as part of a remediation scheme. Secondly, the results show that the amount of heave strain increases with increasing blow counts, but at a decreased rate.

Presented in Figures 6-19 are plots showing the relationship between the amounts of heave recorded, in terms of strain, and the results from CPT testing (normalised cone tip resistance values, q_t) carried out after dynamic compaction. The effect within the Heave Zone of the different material types is discussed in the following sections.

Figure 6-19 shows a strong correlation between the amount of heave recorded and the postdynamic compaction CPT results. Where the CPT results (q_t value) are low, this is typical of an area that has been subject to a significant amount of heave. As the q_t increases, the amount of heave recorded decreases.

There is also a strong correlation between the amount of heave occurring and the number of blows during the dynamic compaction testing. That is, for a given CPT result, the amount of heave strain that has occurred is dependent on the number of blows during the dynamic compaction process. By way of example, for a q_t of 10MPa, the amount of heave strain occurring was generally 2.5% for the area compacted with 16 blows; but approximately 4.5% for areas compacted with 32 blows.



Figure 6-19. Heave Strain vs Post Dynamic Compaction qt

Based on the data presented in Figure 6-19, the amount of heave strain can be related to the normalised tip resistance (q_t) and the number of blows using the following equation:

$$HS = 5Nq_t^{-1.5}$$
 [6-2]

Where HS = Heave Strain (%)

N = Number of blows during dynamic compaction

 q_t = normalised cone resistance (MPa)

Equation 6-2 has a direct relationship between the number of blows and the heave strain incurred, and does not account for the decreasing benefit with increasing number of blows. During the trial works (refer to Chapter 5), it was assessed that the optimal number of blows was 16; hence including a reduced benefit for blows greater than 16, Equation 6-2 becomes:

$$HS = 5(0.8N + 3.2)q_t^{-1.5}$$
[6-3]

Equations 6-2 and 6-3 above provide a general estimation of the amount of strain occurring

within the Heave Zone, however is independent of the material type.

The CPT results have also been used to assess the response of different soil types within the Heave Zone. The amount of heave strain at each increment of the extensometer (between magnets) was calculated and the CPT data were then used to correlate the incremental strain data to different soil types, using Robertson *et al.* (1986) method of soil classification.

The results of the assessment are presented in Figures 6-20 to 6-22. The results have been grouped together based on basic soil types:

- Clays: Robertson *et al.* (1986) classification of Clay (Type 3); Silty Clay to Clay (Type 4); and Very Stiff Fine Grained (Type 4) soils.
- Silts: Robertson *et al.* (1986) classification of Clayey Silt to Silty Clay (Type 5); Sandy Silt to Clayey Silt (Type 6) and Silty Sand to Sand Silt (Type 7) soils.
- Sands: Robertson *et al.* (1986) classification of Sand to Silty Sand (Type 8); Sand (Type 9) and Sand to Clayey Sand (Type 12).

Also presented in Figures 6-20 to 6-22 is the average heave strain, calculated over 1m intervals.





Figure 6-20. Heave Strain of Clay

Figure 6-21. Heave Strain of Silt



Figure 6-22. Strain of Sand

A summary plot of the average heave strain values for the different material types is presented in Figure 6-23, with a summary table of the statistical variation over each interval presented in Table 6-1.



Figure 6-23. Average Strain

Table 6-1.	Summary	of Statistical	Variation	of Heave	Strain
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Depth	Clay			Silt			Sand		
Interval	Count	Average	Standard Deviation	Count	Average	Standard Deviation	Count	Average	Standard Deviation
0 – 1 m	-	-	-	-	-	-	-	-	-
1m – 2m	20	-6.01	5.83	-	-	-	-	-	-
2m – 3m	47	-8.1	3.99	16	-8.78	9.24	13	-4.65	3.84
3m – 4m	44	-9.55	6.10	12	-9.36	8.08	17	-3.20	2.06
4m – 5m	44	-7.23	6.69	11	-5.78	7.39	-	-	-
5m – 6m	64	-0.82	3.45	7	-1.59	1.27	-	-	-

The following is a summary of the influence of the material type on the magnitude of the heave strain occurring:

- There is a large distribution of ranges within the data, particularly for the Silt materials. Considering that Silt is highly susceptible to moisture content variations (which is not measured by the CPT), this is not unexpected.
- There are significantly more Clayey materials than other materials present within the site, which is consistent with classification data from Chapter 3.
- The clay materials exhibit the greatest amount of heave within the Heave Zone.
- The Sand material exhibits the least amount of heave and the Heave Zone extends to a shallower depth (only 4m, compared to 6m for the Clay and Silt materials).

The amount of heave that occurs within the Heave Zone forms a critical part of the assessment of dynamic compaction. Clayey and silty materials can exhibit heave in the order of twice that recorded in sandy materials; however the amount of heave strain in sands should not be dismissed. A heave strain of 4.65% (maximum average heave strain in sandy material) is a significant strain which would need to be considered as part of the dynamic compaction process.

6.2.6 Summary and Conclusions

The results of the review of the measurements within the Heave Zone can be summarised as follows:

- The heave resulting from the dynamic compaction trial can be classified into three types: No Heave, Narrow and Concentrated Heave; and Wide Heave. This is comparable to the types of bearing capacity failure presented by Vesic (1973).
- The lateral extent of the Heave Zone varies between 5m and 9m from the drop location. The extent of the Heave Zone has good correlation to the lateral extent of the general shear bearing capacity failure mode presented by Vesic (1973).
- The depth of the Heave Zone extends to a depth approximately 1.9 times the depth of the craters resulting from dynamic compaction. The depth of the heave zone also appears to extend deeper in clayey and silty materials, when compared to sands.
- The heave strain within the Heave Zone can be in excess of 10%. The amount of

heave within 3m of the ground surface becomes critical for residential developments where the near surface soil characteristics have a significant effect on the settlement of shallow footings. Relationships have been developed, based on using the CPT, that allow estimation of the heave strain (or loosening of the soil) within the Heave Zone resulting from dynamic compaction.

• The amount of heave occurring within the Heave Zone is dependent upon the type of soil within the subsurface profile. Clay materials appear to incur a large amount of heave – approximately double that recorded in sands.

6.3 Review of Settlement Zone

6.3.1 General

As noted in Section 6.1, the Settlement Zone has two main characteristics: the depth of compaction and the magnitude of settlement induced. The settlement induced by dynamic compaction has been the subject of numerous studies; hence a brief review of the literature on these two characteristics is presented in Section 6.3.2.

The subsequent section presents in more detail the results of the trial, with particular focus on estimating the depth of influence of dynamic compaction. The results of the trial are compared to the literature and refinement of the current literature is presented to enable better estimation of the depth of influence of the dynamic compaction process.

In Section 6.3.3, the results of the trial are also used to quantify the effect within the Settlement Zone with further comparisons and refinements made to the literature. Through further refinement of the literature, better estimations can be made of the effect within the Settlement Zone of the dynamic compaction process.

6.3.2 Background

The depth of influence of dynamic compaction has been the focus of studies by Menard and Broise (1975), Luongo (1992), Lukas (1992) and many others. Lukas (1992) presented the depth of influence of the dynamic compaction process to be as follows:

$$D = n (W x H)^{1/2}$$
 [6-4]

where n = empirical coefficient

W = weight of the pounder (tonnes)

H = drop height (metres)

The studies by Menard (1977), Luongo (1992) and Lukas (1992) relate the depth of influence to the pounder weight and drop height but do not consider the effect of the grid spacing.

The influence of grid spacing in dynamic compaction has been the subject of numerical studies by Poran and Rodriguez (1992). Poran and Rodriguez (1992) investigated both the vertical and lateral extent of the influence of dynamic compaction on cohesionless soils and filled ground. They postulated that at each impact location the zone of influence may be represented by a semi-spheroid shape with a (horizontal radius) and b (vertical depth); where a and b are functions of the drop mass, drop height and number of impacts (refer to Figure 6-24 below).



Figure 6-24. Shape of Zone Affected by Dynamic Compaction (Poran and Rodriguez, 1992)

Poran and Rodriguez (1992) presented relationships between the horizontal radius and the vertical depth of the semi-spheroid and the equipment dependent factors as follows:

$$\overline{D}^{=l+m\log}\left(\overline{Ab}\right)$$
[6-6]

where a is the horizontal radius

b is the vertical depth

j, *k*, *l* & *m* are empirical factors

N = Number of drops;

W = Pounder mass (tonnes);

H = Drop Height (metres);

A = Contact area between the tamper and the soil (metres²); and,

D = pounder diameter (metres).

The empirical factors j, k, l and m as presented by Poran and Rodriguez (1992) depend on the relative density of the soil and were found to vary as follows:

- Parameter j: -12.59 to -15.27;
- Parameter k: 6.25 to 8.08
- Parameter 1: -4.43 to -2.49
- Parameter m: 1.90 to 2.32

Chow *et al.* (1994) also investigated the effect of print spacing (distance between drop locations) on ground improvement using dynamic compaction. They presented a different approach to that of Poran and Rodriguez (1992) that estimated the lateral extent of the ground improvement based on assessment of the increase in friction angle in cohesionless materials.

The effect of dynamic compaction has been the subject of numerous numerical modelling studies, whereby the dynamic compaction process has been modelled as a spring and dashpot. Of particular note, Na *et al.* (1998) presented a "simplified" finite-difference method for solving a one-dimensional wave equation, when compared to say Thilakasiri *et al.* (1996). Na *et al.* (1998) presented a stress-strain relationship for the truncated semi-infinite cone model for sand, whereby the engineering properties are related to the changing stress and strain induced by the dynamic compaction process. The results showed good correlation with works carried out at Changi East Reclamation in Singapore, with reasonable predictions of crater depth, and the degree and depth of improvement.

Gu and Lee (2002) developed a constitutive model based on the assumption that improvement is caused by transient effective stress increase induced by pounder impulse rather than vibration-induced densification, which is what Pan and Selby (2002) modelled. The behaviour of the repeated drops was considered by Gu and Lee (2002) to be impulsive, rather than cyclic. Comparison was made by Gu and Lee (2002) between the results of the finite element modelling and centrifuge testing carried out by Oshima & Takada (1998).



The results of the computed and final measured increase in relative density are presented in Figure 6-25 below for three different drop heights and pounder mass combinations.

Figure 6-25. Contours of Increase in Relative Density (from Gu and Lee, 2002)

6.3.3 Depth of Settlement Zone

The depth of the Settlement Zone is assessed by monitoring of the extensometers after the application of dynamic compaction. A summary table presenting the results of the extensometer monitoring is presented in Table 6-2. The results in Table 6-2 have been presented in areas where different number of blows have been applied.

	Depth of Influence (m)					
	Count	Maximum	Minimum	Average	Standard	
					Deviation	
Total	43	13.4	8.9	11.12	1.10	
Dataset						
16 blows	25	13.4	9.3	11.32	1.12	
32 blows	18	12.8	8.9	10.86	1.06	

 Table 6-2.
 Summary of Statistical Variation of Depth of Influence

Examining the total dataset, the results are reasonably consistent with those in the literature. Using the simple form of the equation presented by Lukas (1992), and a weight

of 20 tonnes and drop height of 23m; the empirical coefficient is calculated to vary between 0.42 and 0.62 (with an average of 0.52). These results are consistent with the ranges presented by Lukas (1992) for different soil types – with the average result being consistent with the original estimation of depth from Menard (1977).

The comparison between the depth of influence recorded in areas with 16 and 32 blows show that there is a slight increase in the depth of influence in areas compacted using 16 rather than 32 blows. Further review was undertaken into the influence of material properties on the depth of influence by using the CPT to classify the material over the Settlement Zone (not incrementally). A summary of the different material types is presented in Table 6-3.

	Depth of Influence (m)					
	Count	Maximum	Minimum	Average	Standard Deviation	
Sand	12	13.0	9.8	11.7	0.90	
Silt	-	-	-	-	-	
Clay	31	13.4	8.9	10.8	1.09	

 Table 6-3.
 Summary of Material Type on Depth of Influence

The description of sand, silt and clay is the same in the Heave Zone and is based on Robertson *et al.* (1986) – refer to Section 6.2.5 for more detail.

As can be seen from the results above, on average, the sandy material has a greater depth of influence than the clays, which is consistent with the results from the literature. Using the equation presented by Lukas (1992), the following can be surmised:

- For clays, n varies between 0.61 and 0.46, with an average of 0.55; and
- For sands n varies between 0.62 to 0.42, with an average of 0.51.

These results are consistent with the values presented in Lukas (1992). The results for the sandy material are also consistent with the contours of increase in relative density (presented in Figure 6-25 (a)) by Gu and Lee (2002). As can be seen from Figure 6-25(a), the estimated depth of influence extends down to approximately 11m for dynamic compaction using a 20 tonne weight dropped from a 20m height.

The depth of the grid spacing has also been investigated. As noted previously, limited literature has been published on the effect of grid spacing on the depth of influence of dynamic compaction. Poran and Rodrigues (1992) and Chow *et al.* (1994) postulated that the depth of influence beneath dynamic compaction to be conical in shape and diminishes with distance from the drop location.

In order to assess this theory, the results of the extensometer monitoring have been plotted for different grid spacings. The results are presented in Figure 6-26.



Figure 6-26. Depth of Influence with Grid Spacing

As can be seen from the results, the depth of influence from the dynamic compaction process decreases with increasing grid spacing. Where the grid spacing is larger than 4.5m there appears to be a decrease in the depth of influence achieved from dynamic compaction.

Using the formulae presented by Poran and Rodriguez (1992) and the results of monitoring the dynamic compaction trial, relationships have been established between parameters j and k; and between parameters l and m; which are as follows:

$$j = 3.410k - 1.25$$
[6-7]

$$l = 3.410m - 5.55$$
 [6-8]

The relationships presented above are different to those postulated by Poran and Rodriguez (1992). The main difference identified is that the dataset supporting the work done by Poran and Rodriguez (1992) was predominantly derived from work on cohesionless fill. As can be seen from the statistical analysis presented in Table 6-3, the majority of the dataset consists of clayey material.

The outcome of this is that the grid spacing has a significant effect on the depth of influence achieved by dynamic compaction.

Using the results of the trial, it has been possible to develop a modified equation for estimating the depth of influence of dynamic compaction. It is based on the form presented by Menard (1977) and Lukas (1992) – in that it considered the drop height and pounder weight – but it also considers the effect of the grid spacing. The modified equation for estimating the depth of influence of dynamic compaction is:

$$D = 0.58(WH)^{1/2} - 1.3(S - 4.5)$$
[6-9]

Where D = Depth of Influence (m)

W = Pounder Weight (tonnes)

H = Drop Height (metres)

S = Grid Spacing (m)

6.3.4 Effect within Settlement Zone

The results of the extensometer monitoring have been used to calculate the average strain over the depth of the Settlement Zone. From the results of the extensometer monitoring, the average settlement strain in areas subject to 16 blows is 0.72%; whereas the average strain in areas subject to 32 blows is 1.27%.

In areas where 32 blows were undertaken, and assuming the average amount of settlement strain in the first 16 blows is 0.72%, then this equates to the second 16 blows inducing a settlement strain of 0.55% (refer to Section 5.2 regarding a more detailed analysis about the effect when using 16 and 32 blows).

These strains are relatively small when compared to the amount of strain recorded in the Heave Zone, which is likely to be related to the initial conditions of the uncontrolled fill materials. For example, if the fill is medium dense or better, then it is likely that limited settlement and a large amount of heave would occur.

Notwithstanding this, the results show that the amount of settlement, and strain, diminishes with increasing blow counts. This is consistent with the study undertaken (refer to Chapter 5) comparing the results of the effect of compaction after 16 blows and 32 blows and also what was observed in the Heave Zone.

Presented in Figures 6-27 is a plot showing the relationship between settlement strain, and the results from CPT testing (normalised cone tip resistance values, q_t) carried out after dynamic compaction. The effect within the Settlement Zone of the different material types is discussed in the following sections.





Figure 6-27 shows a strong correlation between the amount of settlement recorded and the post-dynamic compaction CPT results. Where the CPT results (q_t value) are in the order of 4 to 8 MPa, this is typical of an area that was originally soft and has undergone significant settlement during the dynamic compaction process. As q_t increases, the amount of settlement recorded decreases as the material was stiff prior to dynamic compaction.

As with the heave strain, the settlement strain can be estimated as a function of the number of blows and the normalised cone resistance from the CPT. Equations 6-10 and 6-11 present empirical correlations without and with, respectively, considerations for reduction
in improvement for blows greater than 16:

The relationship for the amount of heave strain within an area of dynamic compaction can be estimated using the following equations:

$$SS = Nq_t^{-1.5}$$
 [6-10]

$$SS = (0.8N + 3.2)q_t^{-1.5}$$
[6-11]

Where SS = Settlement Strain (%)

Further consideration has been given to the amount of settlement strain induced for different material types. The amount of settlement strain at each increment of the extensometer (between magnets) was calculated and the CPT data were then used to correlate the incremental strain data to different soil types, using the Robertson *et al.* (1986) method of classifying the soil (refer to Section 6.2.5 for further information).

The results of the assessment are presented in Figures 6-28 to 6-30. Also presented in the figures is the average heave strain, calculated over 1m intervals.



Figure 6-28. Settlement Strain of Clay



Figure 6-29. Settlement Strain of Silt



Figure 6-30. Settlement Strain of Sand



Figure 6-31. Average Settlement Strain

A summary plot of the average settlement strain values for the different material types is presented in Figure 6-31, with a summary table of the statistical variation over each interval presented in Table 6-2.

As can be seen from Table 6-2, there is a large amount of scatter in the data. Considering that the effects of dynamic compaction are heavily influenced by the subsurface profile and the moisture content of the material (for example) this is not unexpected. As such, average values have been presented in Figures 6-31.

It should be noted that at depths below 11m, some movements within the extensioneter were recorded. As these were within the measurement tolerance or the extensioneter (<5mm), it was assessed that these were not a function of the dynamic compaction process. The depth of influence is presented in Table 5-4 for completeness but has not been included in the assessment of dynamic compaction.

The settlement strain is largest within the sand and occurs at a much higher interval than in the clay and silty materials. From the results presented in Table 6-2, the depth of influence of a profile that generally consists of sandy materials also extends much deeper than in silts and clays. This is also consistent with the literature which notes that dynamic compaction is better suited to sandy materials rather than clayey materials.

Depth	Clay			Silt			Sand		
Interval	Count	Average	Standard	Count	Average	Standard	Count	Average	Standard
			Deviation			Deviation			Deviation
4m – 5m	-	-	-	-	-	-	7	4.22	8.78
5m – 6m	-	-	-	-	-	-	4	3.27	4.85
6m – 7m	42	1.97	3.68	7	1.62	2.76	5	0.64	0.37
7m - 8m	48	1.68	2.50	11	0.99	2.02	6	1.22	1.07
8m – 9m	27	2.04	2.17	10	3.40	3.30	5	0.73	0.74
9m – 10m	40	2.58	2.64	12	1.56	2.22	8	0.89	0.58
10m - 11m	28	2.39	3.14	28	1.71	2.02	3	1.01	1.00
11m – 12m	19	1.04	0.67	14	1.13	1.01	2	0.41	0.42
12m – 13m	22	0.56	0.54	13	1.13	1.33	4	0.53	0.17
13m – 14m	21	0.48	0.92	6	0.45	0.59	6	0.45	0.59
14m – 15m	13	0.33	1.01	20	0.12	0.3	7	0.12	0.23

Table 6-2. Summary of Statistical Variation of Settlement Strain

6.3.5 Summary and Conclusions

The results of the review of the data within the Settlement Zone can be summarised as follows:

- The results of the trial generally are consistent with the literature, particularly in relation to the depth of influence of dynamic compaction.
- The grid spacing adopted for dynamic compaction has a significant impact on the depth of influence. Based on the results of this study, a revised formula for estimating the depth of influence of dynamic compaction considering the grid spacing has been postulated.
- Relationships have been developed between the normalised cone resistance, q_t, and the average amount of settlement strain. Further design charts have been presented that relate the amount of incremental settlement strain to the material type.

The amount of settlement occurring is highly dependent upon the type of soil within the profile. As expected, sandy materials respond well to dynamic compaction; with highly variable results found in the clay and silt materials.

6.4 Summary and Conclusions

Based on the results of the study, the Heave and Settlement Zones behave in a different manner during the dynamic compaction process.

The Heave Profile can be reasonably well represented by bearing capacity failure theory presented by Vesic (1973), and generally can be classified as either: No Heave, Narrow and Concentrated Heave or Wide Heave. The lateral extent of the Heave Zone can be reasonably well estimated by the Vesic (1973) bearing capacity failure model.

The depth of the Heave Zone and the amount of heave occurring has not been presented in the literature to date. The depth of the Heave Zone can extend to 1.9 times the depth of the craters resulting from dynamic compaction. The strain that can occur, over a 1m interval, can also be large – up to 10% in a recorded case.

The Heave Zone becomes critical for residential developments, in particular, where the near surface soil profile has a significant effect on the settlement of shallow footings. As such, relationships have been developed, based on the CPT, that allows estimation of the heave strain resulting from dynamic compaction that can be used at other sites.

Treatment of the Heave Zone has generally been undertaken using an "ironing pass" with a low energy impact and reduced drop height (for example, Bo *et al.* 2009). However the design of the "ironing pass" is not detailed in literature, or the design of the number of blows and drop height. As shown herein, the Heave Zone is a crucial part of dynamic compaction and the design of the "ironing pass" needs to be undertaken accordingly.

The Settlement Zone has been the subject of all the literature to date on dynamic compaction. As such, the estimation of the depth of influence and the effect of dynamic compaction within the settlement zone is well documented. A shortcoming of the literature is that the depth of influence presented by most authors does not consider the effect of the grid spacing of dynamic compaction. As a result of the study, a refined method for estimating the depth of influence of dynamic compaction is presented that considers the effect of the grid spacing.

Relationships have also been developed between the overall strain occurring within the settlement zone and number of blows. Further characterisation of the incremental settlement that occurs, over 1m intervals, has been undertaken.

One element of commonality between the Heave Zone and the Settlement Zone is that the ground response is highly sensitive to the material type. Within the Heave Zone, the amount of heave recorded is significantly more in clay than in sand. Conversely, in the Settlement Zone sands have a much higher settlement strain than clays. Therefore, characterisation of the subsurface profile plays a key role in defining the ground response to dynamic compaction.

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

This thesis is based on an industry project looking at the geotechnical remediation of a backfilled quarry for residential and commercial development. The study involved two phases, namely: the characterisation of fill and assessment of settlement characteristics; and the implementation of dynamic compaction to render the site suitable for development.

The first part of the thesis is focused on the mechanism and magnitude of settlement of the fill, in particular, collapse settlement. Collapse settlement is a phenomenon that has been documented around the world. Houston *et al.* (2003) noted that collapse settlement occurs in both natural deposits and man-made fills that have been placed in a loose condition.

The relationships and case histories presented in the literature do not extend to presenting quantifiable relationships between engineering properties (such as density or stiffness) to collapse potential. Authors such as Lawton *et al.* (1989) have noted relationships between collapse potential and density, but did not quantify the relationship. Charles and Watts (2003) and Charles (2003) also noted there was limited applicability of field and laboratory tests to identify collapse potential, as these tests are usually reported in terms of overall strength and compressibility, not collapse strain in relation to the density of the soil.

Further limitations of the available literature also exist as to quantifying the impact of the collapse potential on the ground surface profile, and the development constructed on soils with collapse potential.

Research presented in this thesis into the mechanisms of collapse settlement was undertaken using an electron microscope. Two different samples of the fill material were examined, namely a Silty Sand (SM), and a low plasticity silt or clay (CL). The results indicated that that there was no cementation between particles and that the mechanism involved in collapse was a softening of particle-to-aggregate and aggregate-to-aggregate contacts. It is also likely that some colloidal materials also fill the void space during the inundation process, as was seen around the more stable particle contacts.

Additional laboratory testing was undertaken to quantify the magnitude of collapse strain of the fill material. Two different samples of the fill material were placed in cells at different density ratios and subject to a range of overburden (axial) pressures (between 80kPa and 160 kPa). The samples were then inundated and the amount of collapse settlement was measured.

The results of the collapse testing confirmed that:

- Collapse strain is a function of the overburden pressure and the density of the material.
- The material type has a significant influence on the magnitude of collapse settlement, with sandy materials indicating less collapse settlement than clayey material.
- Relationships were developed that relate collapse strain to the overburden pressure and the dry density ratio.

From the results of further *in situ* testing, statistical correlations were developed between wet density and the dry density ratio; which were then extrapolated to form relationships between the in wet density of the fill to collapse strain.

The research then focused on how to accurately measure the wet density in the field. A method for measuring the *in situ* density of the fill was identified and trialled, namely the DHGD probe. The repeatability and accuracy of the DHGD testing was investigated as part of this research, confirming that the DHGD probe was an appropriate tool for measuring the in situ wet density.

By establishing an accurate profile of density, the amount of collapse strain could then be calculated. Using this information, and the results of research on tunnel collapse by Burland (1995) and Burland *et al.* (2002), it was possible to calculate the effect of collapse on the ground surface profile and also on buildings.

The research showed how it was possible to link the collapse strains from laboratory samples, to field measurements and ultimately assess the likely performance of a residential building constructed on an uncontrolled fill site. After establishing this process, the second part of this research then focused on how dynamic compaction would compact the landform and reduce the amount of collapse settlement occurring.

The available literature on the effects of dynamic compaction has focused solely on the compaction or settlement resulting from dynamic compaction (called the Settlement Zone). Lukas (1992) did investigate some lateral deflections resulting from dynamic compaction; however none of the literature presents information about the near-surface disturbance, or heave, which results from dynamic compaction. The results of full scale trials have been used to assess the effect of dynamic compaction on near-surface materials, or "Heave Zone".

The Heave Zone can be reasonably well represented by bearing capacity failure theory

presented by Vesic (1973), and depending on the ground surface profile can be classified as either: No Heave, Narrow and Concentrated Heave or Wide Heave. The lateral extent of the Heave Zone can be reasonably well estimated by the Vesic (1973) bearing capacity failure model.

The depth of the Heave Zone and the amount of heave occurring has not been presented in literature to date. The depth of the Heave Zone can extend to 1.9 times the depth of the craters resulting from dynamic compaction. The strain that can occur, over a 1m interval, can also be large – up to 10% in a recorded case.

The Heave Zone becomes critical for residential developments, in particular, where the near surface soil profile has a significant effect on the settlement of shallow footings. As such, relationships have been developed, based on the CPT, that allow estimation of the heave strain resulting dynamic compaction that can be used at other sites.

Treatment of the Heave Zone has generally been undertaken using an "ironing pass" with a low energy impact and reduced drop height (for example, Bo *et al.* 2009). However the design of the "ironing pass" is not detailed in literature, or the design of the number of blows and drop height. As shown herein, the Heave Zone is a crucial part of dynamic compaction and the design of the "ironing pass" needs to be undertaken accordingly.

The available literature on the Settlement Zone is primarily focused on the results from a single drop point; however the application of dynamic compaction is undertaken in an interlocking grid pattern with a varying number of blows. The relationship between the grid spacing, number of blows and depth of improvement has not been previously explored but the research presented herein has been undertaken to quantify the effects of changing the grid spacing and number of blows forming part of the dynamic compaction process. As a result of the trials, new relationships are presented for assessing the effect of dynamic compaction and designing the number of blows and grid spacing of dynamic compaction.

Relationships have also been developed between the overall strain occurring within the Settlement Zone and number of blows. Further characterisation of the incremental settlement that occurs, over 1m intervals, has been undertaken.

Also, historical relationships have been presented for the depth and magnitude of settlement resulting from dynamic compaction. Based on the results of a full-scale trial, the previous empirical relationships are refined with amendments to previously established relationships provided to increase the knowledge in the dynamic compaction area.

One element of commonality between the Heave Zone and the Settlement Zone is that the ground response is highly sensitive to the material type. Within the Heave Zone, the amount of heave recorded is significantly more in clay than in sand. Conversely, in the Settlement Zone sands have a much higher settlement strain than clays. Therefore, characterisation of the subsurface profile is critical in defining the ground response to dynamic compaction.

7.2 **Recommendations for Future Research**

7.2.1 General

The research has illustrated that although dynamic compaction has been used since circa 1976 there are still areas where future research can be undertaken. These are described in the following sections.

7.2.2 Collapse Potential

The research has presented a method for relating collapse settlement (or strain) to wet density. The relationships presented herein regarding density ratio and collapse strain is based on limited data. With additional laboratory testing, stronger relationships could be established that relate collapse potential to material specific properties, such as clay content, as well as overburden pressure and density.

A key element of the research presented herein is relating collapse settlement (or strain) to an engineering property that can be measured in the field. However, the measurement of wet density of the subsurface profile, accurately, is arduous.

The DHGD probe is a unique tool, and while it measures wet density it is not commonly available or used as part of geotechnical investigations. Collection of tube samples is often done at 1.5m intervals, which is at a quite coarse sample frequency for the assessment of collapse. Other forms of measuring the in situ density could be investigated; or alternatively other relationships between collapse potential and readily measured engineering properties (such as shear wave velocity) could be examined.

7.2.3 Dynamic Compaction

The research of dynamic compaction has provided additional insight into the compaction process; however a limitation of the research has been isolating and quantifying the effects of the dynamic compaction process in uncontrolled fill.

As noted in Chapter 5, the average settlement strains induced vary between 0.1% and 3% over the depth of influence of dynamic compaction. The settlement strains seem more prolific in loose zones, where settlement strains have been recorded at up to 8%.

The identification of the loose zones prior to undertaking the full scale dynamic compaction trials was a limitation of the research, in that these loose zones could have been specifically targeted during the geotechnical investigation, and monitored during the dynamic compaction works. However, the fill would not be considered "uncontrolled" if the precise construction history was already known.

An area for future research would be to undertake dynamic compaction laboratory trials to confirm and better quantify the results of this research. Laboratory trials could be undertaken and designed to quantify the effects of undertaking loose zones interbedded in a dense subsurface profile.

Furthermore, the soil profile could be better controlled to identify different effects of the materials properties on the depth of influence, settlement strains and also heave strains.

7.2.4 Heave Zone

This research has identified and quantified the effects of dynamic compaction within the upper zone of the soil profile, termed the Heave Zone. Bearing capacity theory has been used to estimate the depth and lateral extent of the Heave Zone; however, considerably more research could be undertaken to better model the extent of this zone.

Furthermore, limited data have been published about the magnitude of the effect within the Heave Zone. Data have been presented from an uncontrolled fill site where the profile is highly variable and the response of the ground to dynamic compaction is also variable. Under controlled conditions there may be an opportunity to better quantify the effect within the Heave Zone.

7.2.5 Numerical Modelling

This research has not addressed the numerical modelling of the effects of dynamic compaction; however through the literature search it was identified that limited modelling has been undertaken.

The available literature indicates that numerical modelling has typically been undertaken using a spring and dashpot model; and limited modelling has been undertaken using ABAQUS. In both cases, the numerical modelling has been examining the effects of dynamic compaction at a single drop point, and typically considered only the Settlement Zone. A field where significant research could be undertaken would be to model the effects of dynamic compaction being undertaken in a grid pattern in different phases (not just a single drop point) and also considering the effects of both the Heave Zone and the Settlement Zone. Within both of these zones, large plastic deformations would need to be considered (up to 10% strains) during the application of the dynamic compaction.

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